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AMERICAN SOCIETY

OF

CIVIL ENGINEERS.

(INSTITUTED 1852.)

VOL. LIV.

PART A.

Being the first volume of the Publications of the
International Engineering Congress,
held under the auspices of the Society,
St. Louis, Mo., October 3d to 8th, 1904.

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1905.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

INTERNATIONAL ENGINEERING CONGRESS, 1904.

Organization and Scope.—The Congress was undertaken, financed and conducted by the American Society of Civil Engineers, at the request of the Louisiana Purchase Exposition. Thirty-seven subjects were selected for consideration, and invitations to contribute papers were issued to specially selected engineers in America and abroad; each of these papers to be a review of progress during the past decade.

Papers and Discussions.—In response to this invitation ninety-seven such papers were received, nearly all of which were printed in advance form and distributed prior to the Congress for the purpose of eliciting discussion. The nationality of the authors of these papers is as follows:

United States, 51,	Holland, 7,	Belgium, 1,	Russia, 1,
France, 18,	Japan, 5,	Canada, 1,	Switzerland, 1.
England, 10,	Austria, 1,	Denmark, 1,	

One hundred and twenty-four additional written communications have also been received, which, together with the oral discussions at the Congress, after revision by the speakers, form part of this Congress publication.

Meetings.—The Congress was divided into eight Sections: Waterways, Municipal, Railroads, Materials of Construction, Mechanical, Electrical, Military and Naval, and Miscellaneous, and twenty-eight sectional meetings were held. There were also two general meetings of the Congress. The total registered attendance was 876, and the average attendance at each sectional meeting about 50.

Publications.—This Volume is one of six containing the Papers and Discussions of the Congress, published by the Society as Parts A, B, C, D, E, and F, of Vol. LIV of *Transactions*. In these volumes although it has not been possible to retain the subdivision by Sections, and no special grouping of subjects has been attempted, the papers and discussion on each subject are grouped. With each volume there is a table of Contents, and the last volume contains an Index covering the entire publication.

Chas. Warren Hunt,

Secretary.

New York, February 25th, 1905.

INTERNATIONAL ENGINEERING CONGRESS.

ST. LOUIS, MO., OCTOBER 3d TO 8th, 1904.

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ST. LOUIS, MO., OCTOBER 3d TO 8th, 1904.

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TRANSACTIONS
AMERICAN SOCIETY OF CIVIL ENGINEERS.

INTERNATIONAL ENGINEERING CONGRESS,
1904.

Paper No. 1.

THE PURIFICATION OF WATER FOR THE PRODUCTION OF STEAM.

By J. O. Handy, Esq.*

In the United States we have all classes of boiler feed-waters, from the nearly pure waters of New England, to the exceedingly hard or "foaming" waters of the Middle West which derive their boiler-incrusting constituents from the limestone, gypsum or other mineral deposits through which they pass.

Corrosive waters containing free sulphuric acid and acid iron salts occur in all coal-producing districts, especially in those places where coal is washed preparatory to coke production. The waters of the Youghiogheny and Monongahela Rivers in Pennsylvania are contaminated in this way, causing great trouble and expense to steam users.

Mountain streams often contain so little scale-forming matter that the excessive amounts of dissolved oxygen or carbonic acid which they carry in solution may cause corrosion. Such instances are familiar to railroad companies operating in the Allegheny Mountains in Pennsylvania.

Peaty waters are found which contain corrosive vegetable acids. Such waters are most common through the Southern States.

* Chief Chemist, Pittsburgh Testing Laboratory.

Well waters containing magnesium chloride are corrosive unless they contain an excess of calcium carbonate.

The more or less complete removal of scale-forming matter or the neutralization of corrosive substances which occur in boiler feed-water has been carried out by several methods in the United States.

These methods may be classified as follows:

I.—Mechanical Methods.

These include feed-water heaters, scum-catchers and blow-off cocks.

II.—Chemical Methods.

a.—Direct Method.—The chemicals are placed in the boiler or run into it with the water supply.

b.—Indirect Method.—The chemicals are fed into the water on its way to a storage tank which serves also for the completion of chemical reaction and for sedimentation.

c.—Intermittent Method.—The chemical treatment is given alternately to the contents of two tanks, allowing reaction and sedimentation to take place during periods of quiet. Partially clarified water is drawn off through filters and repumped to storage tanks.

d.—Continuous Method.—The chemical treatment is given automatically to the water as it enters the apparatus. The chemical reaction, sedimentation and clarification take place simultaneously or successively during the progress of the water through the apparatus.

These methods represent, from chemical and engineering standpoints, the several stages of growth of the art of water softening in the United States.

Unfortunately the less perfect methods are still largely adhered to. This is not always due to lack of discrimination on the part of steam users, but to the limitations imposed by local conditions.

Only legitimate means of effecting some improvement of bad boiler feed-waters will be discussed in this paper. Boiler compounds without logical excuse for existence will be debarred.

Class I.—Mechanical Methods.

Feed-water heaters remove more or less completely from water the carbonate of lime which it contains, but other and more important scale-forming substances are not affected and pass on into the boiler from which it is impossible to remove them except very imperfectly by scum-catchers or blow-off cocks.

It is doubtful whether removal of carbonate of lime from feed-water by means of heaters ever pays. The heater becomes incrusted and continually more unfit for its principal work which is the pre-heating of boiler feed-water. When encrusted with scale it neither purifies nor heats satisfactorily.

The writer believes that the abuse of feed-water heaters for the imperfect protection of boilers is growing constantly less.

Schemes for heating boiler feed-water under pressure before passing to the boiler have never passed the experimental stage, owing chiefly to the imperfect precipitation of scale-forming substances by short exposure. Sulphate of lime deposits as scale in boilers very gradually with increasing concentration. Pressure and temperature have a modifying influence on the rate of deposition, but no temperature is reached in steam boiler practice at which sulphate of lime immediately falls out of solution.

The use of zinc in the form of balls, rods or plates soldered by wires to the boiler shell seems to have prevented corrosion in many cases.* Its field is properly confined to marine practice and a few other cases where chemical treatment is not possible. The *rationale* of its action is the solution of zinc rather than iron due to the electro-chemical relation of the two metals.

Class II.—Chemical Methods.

2 *a*.—Direct Method.

This practice has been and is very general in the United States and the beneficial results obtained have been in exact relation to the judgment shown in selecting the chemicals and the care shown in carrying out the details of the treatment.

* Cary, "The Cure for Corrosion and Scale from Boiler Waters," *Engineering Magazine*, 1897.

The purifying agents used directly in boilers include:

1.—Soda ash,

2.—Caustic soda,

3.—Phosphate of soda (tri-sodium phosphate),

4.—Tannin compounds,

5.—Fluoride of soda,

6.—Aluminate of soda.

Soda ash has been most widely employed. Used without discrimination, it is rarely beneficial.

The experience of the Chicago, Milwaukee and St. Paul Railway, under the guidance of their chemist, Mr. H. E. Smith, proved conclusively that the systematic use of soda ash, combined with regular blowing off of the sludge produced by chemical action, was a measure of great economic importance.

The Soda-Ash System.*

Principle: "When waters are treated in the boiler with soda ash, the incrusting solids are changed to carbonates and precipitated as a soft sludge which is readily blown out, instead of coming down in a crystalline condition and adhering to the boiler."

Amount of Soda Ash Used.

For each grain per gallon.	Soda ash per 1 000 gallons.
Calcium carbonate	0.02 lb.
Magnesium carbonate	0.02 "
Calcium sulphate	0.10 "
Magnesium sulphate	0.13 "
Magnesium chloride	0.16 "

The amount is always limited to a maximum of 10 lb. soda ash per 100 miles run.

Method of Adding Soda Ash.—The soda ash is placed in the boiler of each locomotive when washed and afterward is added at regular times to the water in the tenders.

Sludge Removal.—By means of blow-off cocks drawing from several parts of the boiler, sufficient sludge and alkali are removed

* As developed by H. E. Smith, Chemist, C., M. & St. P. Ry., *Proceedings*, American Railway Master Mechanics' Association, Vol. XXXII, p. 95.

so that the locomotive can be continued in service a much longer time than would be possible otherwise. Some blowing off must be done on the road but the greatest efficiency in sludge removal is secured if time for settling is allowed. About 4% of water is blown out with the sludge.

Results.—The experience of the C., M. & St. P. Ry. Co. extends over many years. In 1899, after 12 years' use of the soda ash process, they reported that for a number of years the results had been uniformly beneficial but it was not until a thorough system of boiler washing was inaugurated and a competent man installed as boiler-washing inspector, whose duty it was to visit all roundhouses, inspect boilers, and give instructions in blowing off and in washing out, that best results were obtained.

Even in the worst district, the life and mileage of flues and fire-boxes were more than doubled.

"While the cost of boiler washing has been somewhat decreased, * * * * the greatest saving has been in the better service and greater mileage of the engines, and the great reduction in boiler work, the boiler makers' pay roll showing a reduction of $75 000 per annum."

Cost of C., M. & St. P. Ry. Treatment.—Waters containing 25 grains of incrusting solids per gallon required not over 4 lb. of soda ash per 1 000 gal. of water used.

The cost of heating the 4% of water which was blown off was estimated at from 0.4 to 0.8 cent per 1 000 gal.

Thus the total expense for chemicals and for loss of heat due to blowing off would not exceed 4.8 cents per 1 000 gal.* The charge due to more frequent boiler washing was not given. The treatment, however, evidently pays well, as it is the standard practice of the C., M. & St. P. Ry.

Other large steam users such as the C. & N. W. Ry. Co., the C., B. & Q. Ry. Co., the U. P. Ry. Co., made extended trials of soda ash and while quite satisfactory results were obtained, they were not sufficiently beneficial to deter these companies from installing other systems of water purification.

Advantages of Direct Soda-Ash Treatment.—Elimination of first cost and interest charges on softening machines or plants.

* The average cost was far less.

Treatment in hands of special men at roundhouses instead of pumpers at isolated stations.

Limitations of Direct Soda-Ash Treatment.—Boilers must be more frequently washed out, because blowing off does not completely remove sludge.

Foaming occurs frequently due partly to suspended sludge and partly to the presence of carbonate of soda in variable excess in the water.

Very hard water cannot be treated sufficiently to prevent scale formation without introducing soda enough to cause foaming.

Conclusion.—While the use of soda ash is a very useful temporary expedient directed toward scale prevention, all large steam users will eventually discard it in favor of methods which purify feed-water before it reaches the locomotive boiler.

Other Chemicals.

Of chemicals other than soda ash, it is probable that phosphate of soda (tri-sodium phosphate) has been most largely used for the direct treatment of boiler feed-waters.

Aside from its price, which is about four times that of soda ash, phosphate of soda has certain advantages. It produces by its action on the lime salts in the water, flocculent, amorphous precipitates which are absolutely non-adherent to boiler surfaces and are easily blown out. Tri-sodium phosphate was first introduced by W. J. Williams, of the Keystone Chemical Co., Camden, N. J., in 1888 or 1889. The sales are said to have been largest between 1892 and 1895.

Like soda ash, it was extensively tried by railroad companies, and if the price had been nearer to that of the cheaper chemical it would have supplanted it.

The writer knows of many instances where it has done good work in improving boiler conditions.

The following are two illustrations:

*Experiments with Soda Ash and Tri-Sodium Phosphate by the P. & L. E. R. R. Co.**—Two locomotives running in freight service between Pittsburgh and Youngstown were equipped with blow-off cocks and tested, one with each chemical. One blow-off cock was

* March 27th–May 11th, 1899.

placed 8 in. above the bottom of the water leg and one at the forward end of the boiler.*

The results of the soda-ash test are as follows:

Distance run without washing out, 4 500 miles.

Quantity of water used, 716 130 gal.

Chemical treatment, March 27th-April 2d, 7 oz. caustic soda per 1 000 gal.

Chemical treatment, April 2d-May 11th, 3.53 oz. soda ash per 1 000 gal.

Cost of soda ash used, 0.25 cent per 1 000 gal.

Cost of soda ash per 100 miles run, 4.1 cents.

Blowing off: 8 to 16 minutes per 135 miles.

Lime in boiler water kept below 3.00 parts per 100 000.

Lime in boiler water, without treatment, after 10 days, 23.6 parts, indicating continuous scale formation.

Soluble salts kept below 102 parts (average).

Foaming did not occur with soda ash, but did with caustic soda.

Some serious flue leakage occurred during use of caustic soda.

During soda-ash treatment no serious leaks occurred.

All leaks were confined to seven flues.

No scale formed on new flues during test. Much disintegrated scale and other sludge was found not to have been removed by the blow-off cocks.

The tri-sodium phosphate test developed similar results.

Scale formation was prevented. The amount of tri-sodium phosphate used was such that the cost was less than 1 cent per 1 000 gal.

The leakage was no greater. The sludge removal was not perfect.

The hardness of the waters treated was equivalent to 5 to 10 parts of carbonate of lime per 100 000. It was, however, mainly due to sulphate of lime.

Neither chemical was adopted for use continuously.

A Tri-Sodium Phosphate Experiment by the U. P. R. R. Co., 1898-99.†—The test was made on the Nebraska Division between North Platte and Sidney and continued 6 months.

The tri-sodium phosphate was dissolved in water and fed through

* See *Railway Age*, June 21, 1899, for description of N. Y., C. & St. L. Ry. Co.'s blow-off cock arrangement on locomotives.

† American Railway Master Mechanics' Association, 1899.

the pumps to the service tanks along the road, in amounts specified by the parties furnishing the chemical.

A careful account was kept of the cost of boiler repairs, boiler washing and of the tri-sodium phosphate used. Comparison with the same period of the preceding year showed as follows:

Decreased cost of boiler work and washing. .	$1 327.05
(The item was $2 860.60 in the preceding year.)	
The cost of tri-sodium phosphate.	1 791.56
Increased cost.	$464.51

The engine-mileage, however, had increased 42.50%, so that on a basis of cost per 1 000 miles there had been a saving of $1.45 or $1 500 per year.

The cost per 1 000 gallons of water treated with tri-sodium phosphate was 4½ cents (tri-sodium phosphate at 6 cents per lb.) It can now be had for less than 4 cents. The results were very good, but were not considered to warrant the expense.

Advantages of Tri-Sodium Phosphate.—Convenience of application. No plant required except a gauged barrel attached to feed line.

Flocculent, non-adherent sludge instead of the more dense, crystalline one produced by soda ash.

Limitations of Tri-Sodium Phosphate.—The price, formerly 6 cents per pound and still about 4 cents, taken together with its high combining weight, makes it at least nine times as expensive as soda ash for water softening purposes.

It cannot be used for complete softening of cold water because the chemical reactions are not finished in any reasonable time without heat. Magnesia precipitates very slowly indeed. Furthermore, the precipitate is bulky and an apparatus with unusual sludge room would be required.

Lime and Soda Ash.—Lime or lime and soda ash (equivalent to wholly or partially causticized sodium carbonate) have been fed directly to steam boilers in some cases.

In the treatment of acid waters from coal mines and washers, some large steam users have kept their boiler water supply neutral by means of milk of lime fed proportionately through feed pumps.

Others have used soda ash alone. The best practice for acid waters, however, is the use of lime and soda ash in equivalent amounts.

The lime treatment leaves sulphate of lime, a scale-forming substance, in the water. The soda-ash treatment leaves free carbonic acid in the water and the iron salts are incompletely removed in consequence. Foaming is also encouraged by the carbonic acid gas.

No by-products of injurious nature are formed when dilute caustic soda, or lime + soda ash are used for acid waters.

*Caustic Soda, Barium Hydrate, Etc.**—In 1880, Dr. C. B. Dudley, Chemist for the Pennsylvania Railroad Company, obtained a patent on a process using caustic soda for purifying water, claiming as a result that the lime and magnesia separate as carbonates and the iron and alumina probably as hydrated oxides.

In 1883, the same author obtained a patent for the use of soda lime for similar purposes. In 1884, Dr. Dudley patented the use of barium hydroxide.

In a recent letter to the author, Dr. Dudley states as follows concerning Pennsylvania Railroad practice: "Under the patents which were granted to us in 1882 and 1883, we at one time operated for about three months at Indianapolis and later for about six weeks at MacDonald, on the Southwest System. No continuous use of these methods of water softening in connection with these patents, has been carried on."

Dr. Dudley then refers to the Pennsylvania Railroad Company's use of soda ash and blow-off cocks, and to the recent installation of three continuous softening plants representing the two most prominent types, the Industrial and the Kennicott.

Sodium Fluoride.—In 1890, Dr. Chas. A. Doremus presented a paper before the American Chemical Society calling attention to the use of fluorides, particularly sodium fluoride for softening hard waters.

At that time the most important statement made was that the precipitation of magnesium "is especially thorough and noteworthy."

Dr. Doremus afterward suggested the use of caustic soda with sodium fluoride.

*M. L. Griffin, "The Comparative Value of Reagents for Removing Lime and Magnesia from Natural Waters for Industrial Uses." *Journal*, American Chemical Society, 1899, Vol. XXI, p. 665.

Griffin* found the precipitating action of sodium fluoride on lime salts in water fairly complete and on magnesia salts very trifling.

The writer† found sodium fluoride an exceedingly unsatisfactory precipitant for any of the lime or magnesia salts occurring in natural waters.

No important use of the fluoride method in water softening practice has come to the writer's attention. Dr. Doremus states that its high price prevents its more general use.

Sodium Aluminate.—In January, 1899,‡ Prof. C. F. Mabery and Ed. Baltzley called attention to the use of sodium aluminate for removing lime and suspended matter from waters.

Some excellent and remarkable experimental results were given.

Griffin's experiments* confirmed to some extent the efficiency of the reagent for precipitating bicarbonates of lime and magnesia and magnesia salts. Sulphate of lime was not affected.

The reagent apparently acts like caustic soda with the assistance of the liberated alumina as a coagulant.

Sodium aluminate has not been used in water softening practice, presumably, because its extra cost was not balanced by sufficient advantage.

Of all the chemicals available for direct treatment of boiler feed-water, sodium phosphate is best and soda ash and lime next. Any direct treatment should be regarded merely as a temporary expedient to be superseded by softening machines as soon as conditions permit.

2 *b*.—Indirect Method.

Treatment of water by the introduction of chemicals into the water as it flows to the storage tank was the first step in the evolution from direct treatment toward complete softening machines. It was designed to avoid the first cost of softening machines by utilizing the existing storage tank for chemical reaction and sedimentation.

Several such plants exist in the United States and many have existed.

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*Griffin (already cited).

† Handy, "Purification of Water for Use in Steam Boilers," *Proceedings*. Engineers Society of Western Pennsylvania, Vol. 15, p. 26.

‡ *Journal*, American Chemical Society.

They have no feature to recommend them except low first cost. In their simplest, crudest form, they consist in:

(1) An arrangement for supplying chemical solution at approximately the required rate from a barrel attached to the suction of the supply pump.

(2) A floating draw-off in the service tank so that approximately clear water may be drawn from it.

(3) A dump valve in the bottom of the service tank for sludge removal. As the tank bottoms are usually flat, only the sludge which lies close to the outlet will be discharged.

Having usually only an imperfect chemical proportioning device, no arrangement for ensuring steady progression of water through the storage tank, and no provision for drawing off more than part of the sludge without emptying the tank, such plants do only imperfect work. The first plant of this type, which the writer knows of, was that of the Union Pacific Ry. Co. at Fossil, Wyo., in use in 1891, designed by Mr. A. Pennell and mentioned in the *Engineering News* of June 9th, 1892.

The general plan has been furthest elaborated by Mr. Howard Stillman, M. E., who designed and installed softening plants on the Southern Pacific Ry. One of these was described in the Proceedings of the American Railway Master Mechanics' Association, June, 1899.*

Mr. Stillman, by means of power from an air or water motor, kept his chemical mixture properly agitated and by regulated air pressure fed it at a predetermined rate into the hard water main.

The mixture was perfected and chemical reaction encouraged by flow through a circulating tank with baffle partitions.

The service tank was provided with several connected holes for drawing off sludge and a floating draw-off for clear water. No arrangement was made to ensure steady and even displacement of water in the tank by the freshly treated water just entering.

Mr. Stillman's devices were the subject of United States patents 595 793 (1897) and 656 331 (1900).

The plan may be criticised in that the proportionate feeding of chemical is dependent on a constant relation of air pressure to water pressure which is an unnecessarily complex arrangement.

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* *The Railway Age*, June 21st, 1899, pp. 13-14.

The plan is commendable for its ingenuity and its provision for many details not hitherto cared for in arrangements of the class.

Any plan utilizing the storage tank for sedimentation must be regarded as an expedient of little more than a temporary character.

2 *c*.—Intermittent Method.

Of the devices already described those operating under method "2 *a*" are "continuous," while those under "2 *b*" are more or less intermittent. In the former case the treated water goes continuously and immediately to the place of use and in the latter there is an interval of uncertain length between its treatment and use.

The devices described under this heading are intermittent in operation in that there is a pause of several hours after treatment. This is to give the time considered necessary for chemical reaction and sedimentation.

The oldest plants of this type in the United States are, the writer believes, those installed in 1894 at Arcadia and Townsend, on the N. Y., C. & St. L. Ry.* Prof. A. W. Smith, of the Case School of Applied Science, was adviser to the railroad officials who put in these plants. In a letter to the writer he thus describes them:

"They consist of the usual large tanks, for treating the water, with mechanical stirrers, flushing gates and the like, and a large tank for making saturated lime water containing similar stirrers. The tanks were provided with surface outflow for the exit of the treated water, which was not filtered.

"The measured volume of water was treated with the calculated volume of clear lime water, then with the proper quantity of soda solution and allowed to settle for ten hours."

These plants apparently did their work well and are still in use.

It will be noted that these earlier intermittent plants followed the Clark process as practised in England, quite closely. Lime water was used, long standing allowed, and filters were not used.

A similar plant was installed in March, 1899, at Calera, Ala., by the L. & N. Ry. Co. from designs of Mr. R. Montfort, Chief Engineer and the Pittsburgh Testing Laboratory, Ltd.

This plant had sufficient tank capacity to handle 300 000 gal. of water daily. It was composed of standard tanks and fittings so

* *The Railway Age*, June 21st, 1899.

that in the event of better water being found the plant could be dismantled practically without loss.

It was in satisfactory operation for about a year. A natural supply of good water was then developed and the softening plant was resolved into the component elements.

The cost of the plant as erected was $2 634.23. The cost of chemicals and operation on a basis of 50 000 gal. daily output was 4.6 cents per 1 000 gal. The output could have been increased without proportional increase in operating expense.

In the commercial development of the intermittent system of water softening, N. O. Goldsmith, Assoc. M. Am. Soc. C. E., of Cincinnati, O., and J. B. Greer, of Pittsburgh, Pa., have done important work. Many of their plants have been installed in the United States and the systems are styled respectively the "We-Fu-Go" and the "Ideal." Their work dates principally from the year 1896.

They added to previous practice in the country the following points:

The use of milk of lime instead of lime water.

The use of sand filters to clarify the softened water.

The "We-Fu-Go"* plants have paddle stirrers while the "Ideal"† plants use compressed air for mixing and agitating purposes. There are no other essential differences.

The general plan of operation is as follows:

Two tanks are provided, the aggregate capacity of which is usually eight times the hourly output expected.

These may be of either wood or iron construction and may be placed on ground level or elevated on trestle-work according to whether repumping of softened water is to be allowed for or avoided.

The tanks are filled alternately to a certain level with hard water. In some plants the milk of lime is added during the filling and the agitator is run at the same time. In most of the plants, the Archbutt-Deeley‡ practice of dissolving the soda ash in the milk of lime and adding both together when the tank is filled, is the one followed.

* Handy, *Proceedings*, Engineers' Society of Western Pennsylvania, December, 1903.
† *Engineering News*, May 26th, 1904.
‡ U. S. Patent 521 522 (1894).

Agitation continues for 15 to 20 minutes followed by a period of perfect rest usually approximating four hours.

At the end of this time the softened water is drawn off from near the surface through floating discharge pipes. It passes through sand or crushed quartz filters to storage tanks from which it is repumped to a higher elevation if necessary.

The largest installations of these plants have been a 3 400 000-gal. "Ideal" plant* at the Tennessee Coal Iron and Railway Company's works at Ensley, Ala., and a 1 440 000-gal. "We-Fu-Go" plant† at the Cambria Steel Works, Johnstown, Pa.

The aggregate daily capacity of plants operating in the United States in 1903 under the plans of Goldsmith or Greer was about 44 000 000 gal.

Both of these gentlemen deserve much credit for their work in extending the knowledge of the benefits derived from water softening.

The Davidson Intermittent Softener.

Mr. G. M. Davidson, Chemist and Engineer of Tests of the C. & N. W. Ry. Co. has devised a water-softening plant‡ which represents probably the best type of intermittent plant.

Its chief features are the use of continuous chemical feed and the adaptation of the supply pump for agitating purposes.

Mr. Davidson adopts the tipping bucket device, already known in the chemical industry and used by Bruun in a different manner for the feeding of the chemical solution to the flowing water.

The bucket as it oscillates under the influence of the stream of hard water is made to operate:

a.—A stirrer in the chemical supply tank (not new).

b.—The plungers of two pumps which feed the required amount of chemical solution into the bucket compartments alternately (new).

The preliminary mixing of chemical solution and water is due to the stirring influence of the hard-water stream entering the compartment after the chemical charge has been delivered into it by one of the pumps. The mixture then flows into one of two 16 by 30-ft. tanks. When the tank is filled, water is pumped from

* *Engineering News*, July 2d, 1903.

† *Engineering News*, May 17th, 1900, Lorain "We-Fu-Go" plant.

‡ *Proceedings*, Western Railway Club, Feb. 17th, 1903.

the top of the tank and forced in through perforated pipes lying in the sludge at the bottom. By this means a thorough stirring and sludge contact are secured, both of which are beneficial. A period of several hours' rest then follows.

The clarified water is drawn off through floating discharge pipes and is repumped to an elevated service tank. Each plant delivers if necessary 240 000 gal. in 24 hours.

The first of these plants was put into operation on July 30th, 1902, by the C. & N. W. Ry. Co. On May 23d, 1904, Mr. Davidson reports 20 plants in use, 17 in Iowa and 3 in Illinois.

The results from these have been so satisfactory that others are to be installed.

Intermittent Softening Plants for Hot Water.

Almost all chemical reactions are hastened by the application of heat. Those which occur in water softening are no exception to this rule. A hot water plant can be operated with much smaller tanks than a cold water one.

The following is a sketch of the method used at one of the most important chemical works in the United States. The writer is indebted to Mr. J. D. Pennock, Chief Chemist of the Solvay Process Co. for this description furnished in 1900. The plant was installed in 1890.

*The Solvay Process Company's Purifying System.**—Onondaga Lake Water is used. The water is hard and saline.

	Parts per 100 000.
Calcium bicarbonate	14.38
Magnesium bicarbonate	1.32
Calcium sulphate	22.88
Calcium chloride	4.90
Magnesium chloride	72.27
Sodium chloride	97.90

Sodium carbonate (soda ash) is the purifying agent used, 25% in excess of the calculated amount being placed in each of two 4 300-gal. tanks before the water enters. The water is at 80° cent. (having been used in condensers). It enters the tanks at the rate

* See also *Transactions*, Am. Soc. Mech. Engrs., Vol. XIII, p. 255.

of 13 000 gal. per hour, which means that three tanks are filled and emptied hourly, making the cycle for one tank 20 minutes.

This plant is interesting because of the small tankage and the high rate of purification. The reaction-tank area is only two-thirds of the hourly output, and there are no mechanical devices to facilitate mixing or hastening the chemical reaction. The high temperature of the water to be treated and the fact that 25% excess soda ash is used, explain the success of the process.

The treated water is pumped through sand filters into the boilers. Seven filters, from 4 to 6 ft. in diameter by 4 to 5 ft. high are used.

The test of the feed-water is regulated to 7 cu. cm. d.n.H_2SO_4 per 100 cu. cm. of water.

After passing the filters the water contains lime salts equivalent to 2.50 parts sulphate of lime per 100 000.

The boiler tubes show a dust-like coating, easily rubbed off.

By blowing off at intervals the concentration of sulphate of soda, carbonate of soda and salt is kept at or below 2° Beaumé (hydrometer test).

One man on each 8-hour shift attends to the treatment.

Summary of Solvay System.—Capacity: per day, 310 000 gal.; per hour, 13 000 gal.; per minute, 216 gal.

Reaction space: 2 × 4 300 = 8 600 gal. = two-thirds of hourly output.

Filters: seven, five of which aggregate 70 sq. ft., surface area.

It will be noted that the reaction tank capacity, instead of being four times the expected hourly output, as is the practice with cold-water plants, is only two-thirds of the output.

2 *d*.—Continuous Method.

The type of machine now referred to is the one which is so designed that the flow of water to the plant operates all necessary mechanism (stirrers, etc.).

The feed of chemicals is regulated by proportioning devices. Proper mixing of chemicals with hard water takes place automatically and the water passes evenly through the machine, while the chemical reaction of softening proceeds to a finish and the mechanical action of sedimentation is almost perfectly effected. A filter at

the top of the machine gives final clarification and the softened water is discharged without repumping into the storage tank.

The perfect action of this ideal softening machine is approached in different degrees by that of each of the several machines of the type which are now in use in the United States.

Historically considered, the Desrumeaux machine* was the first continuous softener introduced into the United States. It was of French origin and was brought to this country by the Industrial Water Company of New York.

It was a development of European practice and evidently had considerable merit as well as some undoubted defects.

Desrumeaux seems to have been the first to utilize by means of a water-wheel the power of the water flowing into the softening machine. He used the power for driving a stirrer in his lime tank. In both lime-water and reaction tanks, he had annular, spiral passageways for the rising water aiming to give it a steady, though circuitous upward movement. Sludge settling on the spiral plates was supposed to slide to outlets properly placed to favor undisturbed fall to the base of the machine. The feed for chemicals was controlled by valves operated by floats in the hard-water box.

Several machines were built, but it has been stated by the Industrial Water Co. that they did not work successfully under American conditions.

The design was modified by Mr. C. H. Koyl and possibly others. Up to date,† railroad and industrial softening plants with an aggregate daily capacity of 8 000 000 gal. have been installed by the Industrial Water Co.

The spiral settling device has been discarded; the small lime-water tank has been replaced by a very large one, and elaborate stirrers have been placed in the reaction tank. The apparatus as a whole consists of from three to four separate tanks, each requiring its own foundation and timbering if elevated.

The lime-water tank is made large, so that it will produce practically clear saturated lime-water of definite strength and not milk of lime of variable lime content. This point is important and is looked after carefully in the design of all good continuous machines.

* U. S. Patent 512 686 (1894).

† June, 1904.

The rate of clarification varies according to whether hard water or soft water is used for making lime water.

Stirring always assists chemical reaction, but the writer does not think that stirring need be very prolonged.

Other machines with no stirring beyond about five minutes' mixing turn out properly softened water.

The course of the water through the apparatus as effected by its internal design is a very important factor in determining the completion of reaction and of sedimentation.

Besides the Desrumeaux, two other continuous water-softening machines have been introduced into this country from Europe. Reference is had to the "Reisert-Dervaux," a German machine brought out by the Automatic Water Purifying Co. of New York, and the "Bruun-Lowener," a Danish machine handled by the American Water Softener Co., Philadelphia.

The Reisert-Dervaux Softening Machine.

This machine has a conical or vase-shaped lime saturator designed to favor clarification of the lime-water. Experience with cone-bottomed but otherwise cylindrical saturators has shown that the vase shape is unnecessary.

The lime- and hard- water flows are proportioned by micrometer valves leading from a receiving box at the top of the machine.

The method of feeding soda solution is by an adjustable siphon which rises and falls with the level of the hard water in the receiving tank. The method of mixing chemicals and hard water is, in the writer's opinion, poor and the internal arrangement of the reaction tank imperfectly planned.

An ingenious, self-cleaning sand filter occupies the upper part of the reaction tank. This device, however, failed to work satisfactorily in an installation with which the writer is familiar and may be criticised on several grounds, in any case. The writer believes there are four or five plants of this type in the United States, two of which are in operation.

The Brunn-Lowener Softener.*

This machine differs from the Desrumeaux in its device for utilizing the water power for stirring and chemical feed. The

* U. S. Patent 583 786 (1897).

Desrumeaux uses an overshot water-wheel and the Bruun-Lowener an oscillating or tipping-bucket. This device has already been referred to when describing the Davidson machine. Bruun has his chemical solutions (milk of lime and soda) in a semi-cylindrical tank placed above the tipping-bucket. The motion of the bucket is caused to actuate a stirrer and also a stopper-valve.

The latter is raised to a definite height by a cam and then seated by a spring. A definite quantity of solution is fed in this way. Mixing of chemicals and water is accomplished by the force of the inflowing water.

Bruun's patent claims are for use of the tipping-bucket as a means of operating a feed-valve in the chemical supply tank. Subsequent users of the device have caused its motion to actuate feed-pumps (Davidson), or to lift and discharge chemical solution by spoon devices (Kennicott).

In the Bruun-Lowener plants built in this country the lime and soda solution is prepared at ground level and pumped to the feed tank at the top of the apparatus. As it is necessary to keep a constant level over the feed-valve, it requires a continuous chemical supply. The hard water passing to the machine drives a circulating pump for this purpose.

It may be said of all tipping-bucket machines that the mixing occurring in the bucket and due to the motion of the inflowing water is not enough to properly inaugurate the reactions which occur in water softening. It is also true that they are not as economical of lime, and, most important of all, the chemical feed is temporarily inaccurate after a period of disuse. This is caused by the settling out of the lime. When the stirrer has worked a few minutes, the feed is normal.

The aggregate daily capacity of the Bruun-Lowener plants in the United States at present is about 1 500 000 gal.

The Kennicott Softener.

The Kennicott water softener* is not of European origin as were the other continuous softening machines just described. Its details were worked out by Messrs. C. L. Kennicott and W. H. Green, while chemists at the Consumers Co. plant at Chicago.

* U. S. Patents 646 108, 665 596, 703 596, 708 717, 717 215, 732 357.

It differs in the following respects from other continuous softeners:

1.—The chemical feed-proportioning device;

2.—The use of soft water for lime-water;

3.—Method of mixing chemicals and hard water;

4.—Means of assisting sedimentation;

5.—Compact, concentric tank arrangement.

Chemical Feed-Proportioning Devices.

The proportioning devices employed in connection with continuous chemical feed in the several softening machines used in this country are:

1.—Weirs (used by Industrial Water Co., also by A. B. Bellows).

2.—Stopper valve actuated by tipping-bucket (Bruun-Lowener).

3.—Pumps actuated by tipping-bucket (Davidson).

4.—Spoons actuated by tipping-bucket (Kennicott *T. B.* type).

5.—Fixed orifices for discharge (Reisert-Dervaux).

6.—Movable and adjustable orifices for discharge (Kennicott *L. P.* type).

1.—*Weir System.*—The use of proportional weirs pre-supposes a smoothly flowing body of water. Mr. Bellows' plan* provides stilling plates for overcoming wave motion.

The Industrial Water Company uses weirs located in a box directly under and receiving the discharge from their overshot water-wheel. Wave motion must influence measurement in such a case.

Fixed weirs serve only for proportioning lime-water to hard water. Mr. Bellows has devised a movable weir (attached to a float) for use with soda-ash solution.

2.—*Stopper-Valve System.*—This plan has already been mentioned. It is for use in connection with a milk of lime and soda feed. A cam actuated by the motion of the tipping-bucket raises the valve and the spring and weighted stem seat it again.

A quantity of chemical mixture, proportionate to the time during which the valve is open, flows into the bucket compartment which has just discharged its contents.

Between runs, lime settles around the valve causing it to stick

* Manager of Pittsburgh Testing Laboratory, Ltd.; U. S. Patent 660 785; *Engineering News*, May 26th, 1904, p. 507.

or feed too much lime before the mixer has done its work. Small bits of wood, sand, etc., may prevent the valve from seating properly and thus the chemical feed will be uneven.

3.—*Pump System.*—Two chemical feed-pumps, having ball valves have pistons connected with a walking-beam actuated by the movement of the tipping-bucket. Judging by published results, Davidson's pumps† work well.

It would be supposed that the alkaline mixture would corrode valves and that it would be difficult to keep the piston packing tight.

4.—*Spoon System.*—This device for use in connection with Kennicott plants up to 500 h. p. is described in U. S. Patent 732 357. A rocker-arm has spoon-like contrivances for its ends. These dip up a definite quantity of chemical solution and discharge it into the tipping-bucket at each oscillation. It would seem much more positive and certain in its action than either stopper-valve or pump.

All tipping-bucket devices pre-suppose a well-stirred chemical mixture, stirrers being operated by the tipping-bucket.

5.—*Fixed Orifice System.*—In this system, orifices in the side or bottom of the hard-water receiving box are controlled by micrometer valves. The depth of the openings under water is fixed but their size may be varied by means of the valves.

The success of this system depends on uniform discharge by the valves which can only be insured by calibration as frequently as experience proves is necessary.

6.—*Movable Orifice System.*—This system is carried out by means of hinged lift-pipes having adjustable openings in their traveling ends.

The pipes are located in the lime-water and soda-solution measuring boxes which are always kept filled to the same depth.

Fluctuations of water level in the hard-water box due to increased or decreased flow of water to be softened are transmitted to the lift-pipes by means of a float and chains.

In the beginning, the discharge slot in the hard-water box is made in exact proportion to the slots in the lift-pipes in the chemical boxes. This having been done, the flow from all of these open-

† U. S. Patent 729 286.

ings is kept proportional by the fact that all of the orifices are at all times under equal depths of water or chemical solution.

Soft Water for Lime-Water.

The object in using soft water is to save lime.

The method of raising soft water to the height where it can be measured and used for making lime-water is a lift-wheel and hollow-shaft arrangement (Kennicott). Soft water has been used for the same purpose at the Southampton, England, municipal softening plant.

It is probable that the influence of soft water on uniform lime-water production is more important than lime economy.

Method of Mixing.

In the Kennicott apparatus the soda-ash solution meets the hard water first. The mixture flows into a revolving hopper having projecting baffles. The lime-water rises constantly and meets the hard water and soda just above the perforated plate which forms the top of the lime saturator.

The completed mixture then turns and flows upward, being very thoroughly stirred, until it passes over into the top of the down-comer cone which is a part of the settling system.

Assistance to Sedimentation.

After the mixing and stirring have been carried out, two things are still necessary:

1.—Completion of all chemical reaction;

2.—Clarification or sedimentation.

For the first result, time and sludge contact are necessary.

For the second, mechanical aids to sedimentation are desirable. In the Kennicott apparatus such assistance is given by a conical bottomless down-comer so designed that the descending turbid liquid constantly loses velocity and drops most of its suspended matter before reaching the bottom of the cone. Settlement of the precipitate thus takes place with the current instead of against it.

The cross-section of the opening of the cone at the bottom is equal to that of the annular space surrounding it at that point, so that the partly clarified water in turning upward does not change velocity. In its further progress, however, its flow becomes steadily slower.

Inclined perforated plates or baffles in this section of the apparatus apparently give the necessary steadying influence and sludge contact.

A wood-fiber filter effects final clarification.

Compact Construction.

The concentric arrangement of tanks or compartments in the Kennicott softener is a great apparent advantage in that it gives:

Protection from freezing.

Ground space and foundation economy.

Continuous Systems for Hot Water.

Any system or plant which fulfills the conditions for softening cold water will necessarily soften hot water.

Steel construction is best and smaller tanks may be used. Nevertheless, it is a mistake to sacrifice anything in thoroughness of mixing or means of securing uniformity of progress of water through the apparatus.

The typical continuous hot-water plant consists simply of:

a.—Soda and lime-water tanks;

b.—Separate feed-pumps;

c.—Mixing tank with baffle partitions;

d.—Sand filter.

The chemical feed-pump may be coupled up with the hard-water feed-pump, but in many cases this is not done, and the only means given the operative to judge of accuracy of feed is a bottle of phenolphtalein solution.

As this reagent gives a pink color as soon as the free carbonic acid in the water has been neutralized, it is absurd to expect it to indicate whether lime enough has been added to decompose bi-carbonates and soda enough for other lime and magnesia salts.

Hot-water softening had best be carried out with an apparatus having more reliable chemical feed devices than proportional pumps.

Adjuncts to Water Softening.

Raymer's Hot Water Changing Device for Locomotives.*

During the use, for steam generation, of waters containing alkali

* A Method of Cleaning and Restoring Boilers to Service Condition. U. S. Patent 757 839.

salts, there is a constant accumulation of these salts, and a corresponding tendency to cause foaming. These salts are incompletely removed by the usual method of blowing off, which is designed primarily only to remove sludge, *i. e.,* material in suspension.

This matter is especially important to railroads having to deal with natural "alkali" waters, but it is also of great interest to progressive railroad officials who have begun to combat hard-water evils by the use of water-softening plants.

All waters containing sulphates, chlorides, or nitrates of lime or magnesia, require soda ash as a purifying agent. This leaves sulphate of soda in the softened water. While such water forms no scale, the tendency to foam or prime increases with concentration and the locomotive has to be sent quite often to the roundhouse to be washed out.

This process as carried out at present on the majority of the railroads in the United States is a time-consuming, boiler-racking ordeal.

The locomotive is out of service about six hours, the heat units in the steam and foul water are all wasted and, as a consequence, fuel has to be used freely for firing up. Worst of all, however, the strains produced by the sudden changes of temperature in the boiler due to the use of cold water for washing out cause frequent flue leakage, etc. This may make calking or other repairs necessary involving more lost time and more expense.

Mr. A. R. Raymer† has devised a plan to overcome, to a large extent, the evils detailed above. His plan is in successful operation at the new shops of the P. & L. E. R. R. Co. at McKees Rocks, Pa.

It consists in the following procedure. On arrival at the roundhouse, the fire under the locomotive boiler is banked. The steam pressure will be about 100 lb. at this stage.

The foul water is discharged through a pipe connection into a "blow-off" tank in the basement of the adjacent power-house. About 50-lb. steam pressure remains in the boiler after all water has been blown off. This pressure is retained. The boiler may be immediately refilled with purified water at 300° fahr. forced in by means of a feed-pump. For the large engines 2 500 gal. of water are required and this amount runs in in from 8 to 10 minutes. The locomotive then has enough steam to start its blower which increases

† Assistant Chief Engineer, Pittsburgh & Lake Erie Railroad Co.

the fire so that in a few moments sufficient steam pressure is available for immediate duty.

Thus in from 30 to 40 minutes locomotives have their boiler water completely changed and are ready for service. This is done without sudden cooling or heating, or the strains and leakage which that entails.

There is a saving also of at least 2 500 000 B. t. u. representing the super-heat in the foul water of one locomotive.

The most important result, however, is the great saving of time during which the locomotive may be in service.

The arrangement of tanks and piping and the routine of the entire water-changing scheme is readily understood from Mr. Kaymer's patent, which has been already referred to.

The device is exceedingly valuable to the P. & L. E. R. R. Co. and deserves general application.

The Economic Results of Water Softening.

The considerations which lead to the taking up of water softening by steam users may be grouped as follows:

First.—Loss of service of locomotives or boilers, due to impossibility of satisfactory, continuous operation with hard water.

Second.—Possibility of substantial savings in fuel and repair bills and the checking of rapid deterioration of boilers.

The cost of water softening undertaken for the first reason is not always a matter of prime importance. It must be reasonable, of course, but results are the main thing. In most cases, however, water softening stands or falls by the relation of what it costs, to what it accomplishes.

The charges against a water-softening installation are:

Interest on cost of plant.
Depreciation.
Chemicals for softening.
Attendance.
Power for operation (and repumping).

The credit items for a softener are:

Fuel saving.
Repair saving.
Depreciation saving.
Increased service obtainable from steam generators.

Cost of Softening Plants.

The best softening plants cost from $4 to $5 per h. p. for installations up to 1 000 h. p., for 1 000-2 000 h. p., the cost is $4 to $3 per h. p. From 2 000-5 000 h. p., the cost is $3 to $2 per h. p. From 5 000-15 000 h. p., the cost is $2 to $1.20 per h. p.

Depreciation.

The above figures refer to steel construction. Plants for which wooden tanks are used are sometimes offered at as high rates, but the depreciation of wooden tanks under service conditions is much greater beside other disadvantages. If steel tanks are reasonably well cared for, 5% is more than enough to allow for their depreciation.

Cost of Chemicals.

The quantity of chemicals required for softening varies directly with the character of the water treated. The prices of lime and soda ash do not vary much in different sections of the country. There is, however, great choice in commercial lime, much of the building lime being so high in magnesia as to make it unfit or uneconomical to use. From 90 to 95% lime can be had and should be insisted upon.

The cheapest waters to soften are those the hardness of which is due to carbonates of lime and magnesia only. Such waters require simply lime-water treatment. It costs only 0.2 cent per 1 000 gal. to remove 1.42 lb. of carbonate of lime (equivalent to 10 gr. per gal.) and only 0.48 cent to remove the same quantity of carbonate of magnesia.

These amounts are sufficient to give a great deal of trouble in heaters and boilers

The removal of sulphates and other soluble compounds of lime and magnesia from water requires the use of soda ash. It costs 1.20 cents per 1 000 gal. to remove sulphate of lime equivalent to 10 gr. per U. S. gal. The same amount of sulphate of magnesia requires 1.36 lb. of soda ash which costs 1.36 cents per 1 000 gal.

The cost of chemicals for softening water varies from 0.5 cent to 5 cents per 1 000 gal., averaging from 1 to 2 cents.

Attendance.

The cost of attendance at softening plants varies greatly, but is never more than that of the time of one man (or boy). It is

often much less. The most common arrangement is for the engineer or pumper to look after the softening plant.

With the best type of plants two or three hours per day are all that are required for attendance unless the installation is very large.

Chemical tests for control of the softening plant can be carried out by persons of average intelligence.

Power for Operation.

Separate power installations for softening plants show bad engineering. Such plants are absolutely dependent on outside power, and such small steam installations are costly to operate.

The best plants have all stirrers or other mechanism actuated by water power. The flow of water to be softened starts everything.

Repumping.

Many softening plants now in existence are so designed that the softened water is discharged at or near ground level. This involves the use of a pumping plant to re-elevate the softened water.

The best type of softeners are those built in the form of towers and discharging the softened water into service tanks at a level only a few feet below the point at which the hard water was received.

Fuel Saving.

The earliest recorded researches on the subject of loss of heat caused by boiler incrustation are those of John Graham, conducted in 1850 to 1857 and published in the Memoirs of the Literary and Philosophic Society of Manchester in 1860. He says: "A scale of sulphate of lime $\frac{1}{16}$ in. thick reduced the efficiency 14.7 per cent."*

Dr. J. C. Rogers states that a scale $\frac{1}{16}$ in. thick makes 15% more fuel necessary than with clean heating surfaces.†

Rankin states that it is estimated that $\frac{1}{16}$ in. of scale requires the use of 16% more fuel.‡

Reuben Wells ran two engines for several months with heating surfaces covered with scale and an equal length of time later with

* Clark, "Steam Engine," p. 218.

† Rowan's "Boiler Incrustation and Corrosion."

‡ "Steam Engine."

clean heating surfaces and found a fuel saving of 17½% with clean boilers. Details of scale thickness or density are not given.*

Tower cites the case of a boiler operated at the "Conservatory," France. When clean, 8.50 kilos of steam were produced per kilo of coal. When incrusted only 3.87 kilos of steam were generated per kilo of coal. The efficiency of the fuel was reduced 56 per cent.†

These records would seem to be sufficiently convincing and yet they are sometimes questioned as if they were theoretical calculations instead of actual results.

The National Association of Stationary Engineers undertook a series of experiments‡ on the "Conductivity of Boiler Scale." These experiments covered the rate of transmission of the heat of steam at 212° fahr. through brass, iron, boiler scale, plaster of paris and Portland cement.

It having been found that a sample of boiler scale conducted heat at about the same rate as plaster of paris, a tube coated with the latter was used for further experiments.

Such a tube was fastened into a cylindrical vessel so that the two ends projected from top and bottom. Water in definite amount was placed around the inner tube and heated gases from a burner with constant gas supply were passed up the middle tube. It took no longer to heat the water to 205° fahr. when the heating surface was coated with plaster than with a clean tube.

A cement-coated tube allowed the surrounding water to be heated to 205° fahr. in 19 minutes, 23 seconds, while the clean tube gave the same result in 16 minutes.

It is pointed out that the efficiency in this case was far from being in the ratio of 4:71 as the relative conductivities of cement and iron would lead one to expect.

The final rather remarkable conclusion is reached that a $\frac{3}{32}$-in. coating of scale would make no appreciable difference in the efficiency of a well-designed boiler.

The fact is overlooked that all boiler scale is not comparable with either plaster of paris or Portland cement coatings and furthermore, that experiments carried out at boiler temperatures

* *Proceedings*, American Railway Master Mechanics' Association, Vol. X. p. 139.

† "Useful Things to Know About Steam Boilers." p. 87.

‡ Reported in *Power*, May, 1896.

and under boiler conditions would probably show much greater interference with heat efficiency.

Kent* says that the amount of loss of heat due to scale deposits is often overestimated.

The loss depends upon the kind of scale as well as upon its thickness, but it does not increase directly with thickness.

Kent says that he once made a test of a water-tube boiler, the tubes of which were coated with a scale $\frac{1}{4}$ in. thick. He obtained practically the same evaporation as he obtained a few days later when the boiler had been cleaned. The scale was soft and porous.

Results such as these should make writers on water softening more temperate in their estimates as to the effect of scale of increasing thickness, but they do not detract from the value of the records already quoted to the effect that $\frac{1}{16}$ in. scale may cause a loss of as much as 15% of fuel, especially when the boiler is worked hard.

The following concrete instances have a direct bearing on the question of fuel saving due to keeping tubes clean by the use of softened water.

A "Kennicott" Plant at an Ice Factory.—The Everett Audit Company of Chicago certified under date of September 10th, 1900, that they found 16.63% less fuel required per ton of block ice produced by the Consumers' Company after installing a water softener.

Fuel saving at the above rate with coal at $1.35 per ton meant an economy of $4 950 per year. Lake Michigan water which was used in this case contains 12.16 parts of scale-forming solids, chiefly carbonates, per 100 000 and is classed as fair for boiler use.

An "Industrial Plant" at a Chemical Works.—Mr. John Metz, Superintendent of the Nichols Chemical Company's works reported 20% less fuel required with softened water. They used 2 500 h. p.

Fuel Saving on Railroads.—Three railroads, the Union Pacific Ry., Chicago & Northwestern and the Pittsburgh & Lake Erie R. R. Co. have installed a large enough number of softening plants to warrant the expectation of definite results showing fuel economy, etc.

Of these only the Union Pacific Ry. Co. has given out prelimi-

* "Steam Boiler Economy," p. 318.

nary results. They report 7½% increase in gross ton-miles per pound of coal. This is believed to represent the effect on boiler conditions of 10 softening plants on the Nebraska Division and is really not a comparison between steam raising with clean tubes and the same with scaled ones.

Better results are sure to be obtained. Even as they are, they lend additional evidence to an already certain matter.

Important evidence as to fuel saving due to scale prevention is sure to be available during the present year.

M. H. Wickhorst, Engineer of Tests of the C., B. & Q. Ry., estimates a fuel saving of $150 per locomotive per year due to water softening.

Boiler Repair Saving.

Estimates.—For C., B. & Q. Ry. Co. engines operating in Illinois and the hard-water districts of the Middle West, Wickhorst estimates that boiler repairs cost $1 200 per locomotive per year, and that softening should reduce this at least 25 per cent.

The engineering department of the P. & L. E. R. R. Co., having moderately hard waters to deal with, estimate that with water softening a saving of at least $100 per locomotive per year will be realized from boiler repair saving.

Actual Results.—Returns from railroads, etc., having water softening in operation give results which exceed the above estimates.

The C., M. & St. P. Ry. Co. reported in 1899 that Smith's soda-ash treatment of boiler waters had made possible an annual saving of $75 000 in boiler makers' pay roll.

The Union Pacific Ry. Co. report as a result of water softening 34% decrease in boiler repairs per engine-mile.

The C. & N. W. Ry. Co. report 40% decrease in boilers makers' force required for repairs on their Iowa Division during the last six months of 1903 as compared with the corresponding period in 1902.

The P. & L. E. R. R. Co. report that on comparing roundhouse conditions in May, 1903, with those in May, 1904, they find that in 1903 six men were constantly engaged in repairing serious flue leakages. Twenty-eight locomotives required attention daily.

Now two men have an easy time calking comparatively trifling leaks on perhaps thirteen locomotives daily.

The Consumers' Co. in Chicago now require no scale removal or boiler repairs due to bad water. This means a saving of $780 per annum at an 800-h-p. plant.

The Nichols Chemical Co. report 75% saving in boiler repairs due to softening a moderately hard water.

Depreciation of Boiler Plant.

It is difficult to put into dollars and cents exactly what water softening means in prolonging the life of boilers. The wear and tear on boilers using softened water would certainly average 75% less than with untreated water.

The U. P. R. R. Co. now finds a set of flues lasting two and one-half years, where formerly six months to a year was the limit.

On the C. & N. W. Ry., locomotives formerly requiring to have flues reset every three or four months, now run for a year or more without this attention.

Washing Out Boilers.

A marked decrease in this item is noticed by users of water-softening plants.

While the accumulation of soda salts necessitates water-changing by blowing off quite frequently, yet complete washing out for the removal of sludge and scale is much less frequently required than formerly.

On the P. & L. E. Ry. the periods between wash-outs have been extended to 20 days for passenger and 45 days for freight engines. These intervals will probably be still further increased.

The C. & N. W. Ry. Co. report that engines do not need washing out as frequently with the purified water as they did without.

During the last seven months of 1903, the average mileage between wash-outs of locomotives on the Nebraska Division of the Union Pacific Ry. was more than doubled. This meant a considerable reduction in labor and cost of water for washing out, as well as increased life of boilers due to less frequent cooling down and heating up again.

Increased Service from Locomotives.

When locomotives are in the roundhouse or shop, their owners are losing:

1.—Interest and depreciation.

2.—Their entire earning capacity.

The P. & L. E. R. R. Co. estimates Item 1 to mean in their case at least $23 000 per year.

Item 2 would mean a much larger figure in all cases where business is lost owing to lack of sufficient available motive power. Such cases have been numerous in the last few years.

Instances are known in industrial plants where water softening has made so much more boiler horse-power constantly available that contemplated extensions of plant have become unnecessary.

It is already evident that the financial benefits derived from the adoption of water softening far outweigh the expense.

Chemical Control of Softening Plants.

Skilled supervision is not necessary after the operative has learned to make tests for hardness, alkalinity and excess of lime.

Hardness is determined by the soap test, alkalinity or acidity by titration, with standard acid or alkali. The silver nitrate test is used for detecting excess of lime.

By the use of tables the operative is able to change the treatment to suit the conditions shown by his tests.

Pfeifer has made a number of very valuable suggestions concerning methods of analysis for water softening purposes.*

Proctor† and Tatlock‡ and McGill|| have all contributed to the better understanding of the chemistry of water softening.

The next few years will see further developments in the art of water softening and a much wider appreciation and realization of its benefits.

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* "Untersuchung und Reinigung des Kesselspeisewassers," *Zeitschrift für Angewandte Chemie*, 1902.

† "Some Recent Methods of Technical Water Analysis," *Journal*, Society of Chemical Industry, 1904. p. 8.

‡ *Journal*, Society of Chemical Industry, 1904. p. 428.

|| *Journal*, Society of Chemical Industry, 1904. p. 516, and earlier.

TRANSACTIONS

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INTERNATIONAL ENGINEERING CONGRESS,

1904.

DISCUSSION ON

THE PURIFICATION OF WATER FOR THE PRODUCTION OF STEAM.

By Messrs. L. M. Booth, A. McGill, E. H. Peabody, F. B. Leopold, Hans Reisert, G. M. Campbell, Fred. J. Miller and J. O. Handy.

L. M. Booth, Esq., New York City. (By letter.)—Referring to Mr. Handy's implied criticism in regard to the length of the stirring period, experience has shown that in wide settling tanks with their slow upward flow two hours is generally sufficient for "complete" subsidence of precipitates, especially where the new precipitate is brought into contact with older precipitates, as in the case of an "up-flowing" stirred reaction tank. A measurably longer period of time is required for "complete" reaction. Arguing from these premises, a chemist will scarcely deny that the surplus time necessary for the latter is more profitably employed in stirring than in subsiding. Mr. Booth

Mr. Handy's injunction in regard to the use of lime which is high in calcium oxide is eminently proper. It happens frequently, however, owing to local conditions and transportation facilities, that pure lime cannot be obtained without undue expense. As a notable instance of this, the limestone of Cuba contains a very large percentage of magnesia and other impurities. Here the use of a lime tank of large diameter is of especial value, as it permits the use of local lime with economy and without prejudice to the results obtained. Clear saturated lime-water is produced in such a case, even when the plant is operated at the slow rate of 5% of its capacity.

A. McGill, Esq., Ottawa, Canada.—The author states (page 4) "these methods represent, from chemical and engineering stand- Mr. McGill

Mr. McGill. points, the several stages of growth of the art, etc." If this statement refers to historical progress only, there can be no objection to it. If it implies the superiority of continuous over intermittent methods, by virtue of their continuous action, it must be called in question.

Doubtless there is a fascination about any apparatus in which hard water continuously flows in at one end, and soft water is delivered, with the same rate of flow, at the other end. But unless the capacity of the treating and settling tank between the inlet and outlet ends of the apparatus is very large indeed in comparison with the volume of water delivered, an important chemical principle is ignored, *viz.*, the influence of the time factor in determining chemical reactions.

This influence, so far as water softening is concerned, has been studied by Pfeifer* and others. We are necessarily dealing with very dilute solutions; and their tenuity is constantly increasing as the lime and magnesia fall out of solution. We know that the rate of chemical change is lessened as the quantity of matter dissolved becomes smaller (law of mass action). That a very appreciable time is required to complete the reactions desiderated in water softening is known to every chemist, and any one may demonstrate the incompleteness of these reactions in cold, dilute solution by setting aside, in a clean glass vessel, a quantity of water which has been correctly treated, and after a period of, say, six hours, seems perfectly clear. A further deposit of lime and magnesia will certainly be formed after another period of six hours.

Intermittent systems of treatment take account of the time factor. It is true that the so-called continuous systems may make practical recognition of this time factor by using a very large settling tank, thus insuring an interval of many hours between the inflow and the outflow of a specific mass of water. But the special advantages (economy of space and reduced first cost) claimed for these systems disappear in such case; and the writer's opinion is that not only efficiency but economy will be found on the side of intermittent systems of treatment.

Mr. Handy makes the following statements: On page 8

"Very hard water cannot be treated sufficiently to prevent scale formation, without introducing soda enough to cause foaming"; and, on page 25

"During the use, for steam generation, of waters containing alkali salts, there is a constant accumulation of these salts, and a corresponding tendency to cause foaming. These salts are incompletely removed by the usual method of blowing off, which is designed primarily only to remove sludge."

* *Zeitschrift für angewandte Chemie*, 1902, p. 193.

These points are well taken. The evil of scaling was quite Mr. McGill. naturally the first to receive attention by railroads; but experience has shown that foaming may become an evil of scarcely secondary importance. Foaming is a property whose causes are not fully understood. It is known, however, that the presence of soda salts above a certain amount will cause a water to foam, and especially so if suspended matter is present.*

The amount of soda salts which may be present without making the water impracticable is perhaps open to question. The writer is pretty well convinced that when the equivalent of 300 parts sodium oxide per million is in solution, the water will be found unfit for boiler use. For purposes of the present argument it will be convenient to assume this limit.

Since soda salts naturally present in water cannot be removed by any precipitation method, we must regard water which naturally contains more than 300 parts soda per million as hopelessly unfit for use.

Soda, whether used as soda ash or as caustic, necessarily remains in solution in the treated water. It follows that a natural water containing 300 parts soda per million is not amenable to any soda treatment; that a raw water containing less than 300 per million ($= a$) may be treated with soda up to ($300 - a$) parts per million, while a water which is naturally free from soda salts may receive a maximum treatment of 300 soda (Na_2O) per million.

300 soda per million (used as soda ash) is capable of reducing 271 parts of permanent hardness due to lime; or (used as caustic soda) 194 parts of magnesia as permanent hardness. Expressing permanent hardness ($H\ p$) in terms of lime (CaO), we may say generally that permanent hardness to the amount of $\frac{56}{62}$ ($300 - a$) parts per million is reducible by means of soda ($a =$ soda naturally present).

For the removal of permanent hardness due to sulphates, the only reagent other than soda which is likely to become practically available is barium hydrate. It is desirable to consider what we can afford to pay for this article.

When a natural water of high permanent hardness, due to sulphates, is already loaded up with soda to such an extent that further addition of soda is out of the question, the price which we can afford to pay for barium hydrate is only limited by the cost of using the water without treatment, or of finding another supply. Such extreme cases will not be considered, although they are far from unknown to railway men. But it often happens that a soda treatment, while rendering it possible to use a bad water, leaves it still an undesirable and troublesome supply. In such cases, a treatment

* McGill, *Canada Electrical News*, January, 1904.

Mr. McGill. in whole, or in part, by barium, might make it a good water. Adopting the limit for soda already mentioned, let us consider the cost of treatment up to this limit.

300 parts of soda (Na_2O) suffices to reduce 271 units of permanent hardness (one unit, $H\ p = 1$ part, CaO, per million). If we take soda ash at 1½ cents per lb. of true carbonate, and caustic soda at 2½ cents per lb. of true hydrate, this means a cost of 7.7 cents per 1 000 gal. when the hardness is altogether due to sulphate of lime, or 9.7 cents per 1 000 gal. when due to sulphate of magnesia. Since sulphate hardness is usually due to both lime and magnesia, it will be fair to assume 8.5 cents per 1 000 gal. as an outside cost for soda treatment.

In order to accomplish the same result with barium, 7.4 lb. of the oxide, equivalent to 15.2 lb. of the crystallized hydrate ($Ba(OH)_2 + 8\ H_2O$), would be necessary. This means that we should have to buy barium oxide at 1.15 cents per lb. (= $23.00 per ton), or barium hydrate at 0.56 cent per lb. (= $11.20 per ton) in order that the worst cases of permanently hard water now capable of treatment by soda should be capable of treatment with barium at no increased cost.*

The lowest price at which barium oxide by German manufacturers has been offered to the writer is $78 per ton (= $37.90 per ton of crystallized hydrate) laid down in Montreal. The United Barium Company of Niagara Falls has made somewhat lower quotations (about $60 per ton for the oxide, or $30 per ton for the hydrate), but it is certain that the article must be supplied at much lower prices than now obtain before it can command an extensive sale for softening feed-waters, having reference to such waters as can now be made at all tolerable by the use of soda.

The writer is, however, of opinion that the time is not far distant when barium will be practically available. Large deposits of heavy spar exist within reasonable distance of the regions where the greatest demand for barium as a softening reagent exists. Water power suited to the development of electric energy is plentiful, and the immense consumption of the article which would surely follow its cheap production would insure the most favorable conditions for manufacture at low cost.

Mr. Peabody. E. H. Peabody, Esq., New York City. (By letter.)—On page 26 of this interesting and valuable paper, the author states that the presence of sulphate of soda in the softened water, while causing

* It is proper to notice here that when barium hydrate is used to remove permanent lime hardness an equivalent amount of lime is set free and becomes available for reducing temporary hardness. Where the temporary hardness is due to bi-carbonate of lime this means that every 153 parts barium oxide used (= 315 parts crystallized hydrate) will not only take care of 56 units of permanent hardness, but also of 56 units of temporary hardness, which under a soda treatment would have required the addition of 56 parts more lime.

no scale, has a tendency to make the boiler (locomotive type) foam or prime, and this tendency increased with concentration. Mr. Peabody.

The writer desires to ask Mr. Handy if he knows of any cases where the presence of sulphate of soda, or of soda ash, in the feed, has caused foaming or priming in the boilers of standard stationary type, particularly water-tube boilers.

The writer's experience, based on some experiments made in California for the express purpose of investigating this point, showed that the presence of sulphate of soda in considerable concentration, the boiler having been under steam 33 days, did not cause any foaming whatever, although the boiler was forced to about double its rated capacity. During these experiments, soda ash was added to the feed tank in considerable excess, with the object of noting the effect of this chemical on the quality of the steam. The steam remained practically dry at all times. The boiler used was of the water-tube design of well-known make, and the feed-water had been purified by one of the lime-soda methods of the "continuous" type.

The paragraph noted in the author's paper evidently refers to the system where the chemicals are mixed with the feed-water and fed directly to the boiler, and it is possible that this creates new conditions which would cause the foaming.

Any other information, however, which the author can give upon this point will be of great interest.

The writer would also be glad to understand how the saving of 2 500 000 B. t. u. noted on page 27 is effected? Does the escaping foul water partially heat the clean water, or is some other method used?

F. B. Leopold, Esq., Chicago, Ill. (By letter.)—The subject of water purification is a very interesting one, and the purification or softening for steam purposes especially so; and, as a brief history of what has been accomplished in this work in the United States, Mr. Handy's paper should certainly be appreciated by all, because it evidences a vast amount of time and care devoted to the securing of reliable data, and little can be added to it as a record of the progress made. Something should be added, however, in one or two instances, and credit given more fully where it is deserved. The writer's rather intimate connection with this branch of the business of water purification since 1895 seems to justify this assertion and the further statement that he cannot fully agree with Mr. Handy in all of his conclusions. Mr. Leopold.

Mr. Handy rather conveys the impression that the plants established by the New York, Chicago and St. Louis Railway at Arcadia and Townsend in 1894, and by the Louisville and Nashville Railway at Calera, Ala., in 1899, were the earliest built in this country, and follows this with a statement giving N. O. Goldsmith,

Mr. Leopold. Assoc. M. Am. Soc. C. E., of Cincinnati, Ohio, and Mr. J. B. Greer, of Pittsburgh, Pa., credit for some important work in the commercial development of the art. The writer is well acquainted with both these gentlemen and has no desire to take from either in the smallest degree all the credit due. In fact he desires to emphasize Mr. Handy's remarks as to the credit to which both are entitled, but they cannot be linked together. Neither does he think that Mr. Handy has quite reached the date of the first installation. In 1891, H. Waterbury & Sons, Oriskany, N. Y., installed a plant, which is the earliest in the writer's knowledge. In 1895, Mr. Hand, of Des Moines, an engineer, advised the writer that he had installed two or three plants a couple of years previously. However, it was either in the autumn of 1894, or early in 1895, that the We-Fu-Go Company, of which Mr. Goldsmith was Vice-President and Manager, installed its first plant as a business proposition; and from that date until 1898 he was practically alone in the field, working against the almost universal prejudice of both engineers and chemists; and he, therefore, deserves to stand alone as the pioneer in the water-softening business in this country on the present lines. Strange as it may seem in the light of the present universal favor among engineers and chemists of water-softening plants, in those early days scepticism, and very often ridicule, were encountered. Many engineers eminent in their profession refused absolutely to accept it as practical for commercial purposes. Nor was this scepticism confined to engineers. Many chemists when called in consultation ridiculed the idea, and, for the first two or three years, practically every plant sold was installed not only on a guarantee as to results, but on an agreement to remove the plant and replace all connections at the cost of the company installing it. In this way only, on the absolute merits of the process as demonstrated by actual operation, was recognition secured from the Engineering Profession in general. In this fight, Mr. Goldsmith stood alone and is, therefore, entitled to the fullest share of credit.

In the description of these We-Fu-Go plants, Mr. Handy is correct, but in the use of lime and soda he is somewhat mistaken. The Archbutt-Deeley process of dissolving the lime and soda was not used, as experience proved that it was not as economical or satisfactory as using separately. The practice was, and is to-day, the use of lime first and after the lime reaction to add the soda ash.

Nor can the writer agree with Mr. Handy that the best type of intermittent plant is one that depends on sedimentation alone, and depends on proportional pumps for chemical feed, as he has seen many waters that after 24 hours' sedimentation would still retain enough floating matter to leave an objectionable amount of sludge in boilers. Chemistry depends in its every process upon the most

minute accuracy of its every act. As water softening is a purely chemical proposition, the greater the accuracy the more perfect the results; and for this reason the intermittent plant, in principle, as designed and introduced by Mr. Goldsmith, and as still used by a number of builders, is the most perfect of this type: First, because it is the only plant in which absolute accuracy of treatment is insured; second, because the water is all filtered after treatment and sedimentation, insuring the removal of practically all the sludge which would otherwise deposit in boilers. The power required is very insignificant and in the writer's experience (since 1896), during which time he has been intimately connected with the installation of probably 125 plants, he has never found it necessary to install an independent steam plant. Nor is he ready to admit that the continuous plant is the ideal one, although the first designs of this type of plant in this country were also made by the We-Fu-Go Company, under the joint supervision of Mr. Goldsmith and himself. The assertion is, however, made that there are many situations in which it has the advantages so greatly in its favor as to bar from consideration the other type for that particular situation. The ideal machine is one that combines in its construction all the elements necessary to produce in its product an output as nearly perfect and uniform as possible under all conditions. This definition covers the intermittent system, utilizing individual treating tanks with agitators and filters. The mechanical arrangement of these may add to or detract from the efficiency, convenience or economy of operation. Mr. Leopold.

There are some conditions under which the ideal must give way to the practical; and, based on the writer's experience, the conditions which make preferable one or the other type are about as follows:

The advantage is greatly in favor of the intermittent plant where the capacity is less than 50 000 gal. per day, in all cases where space permits;

For any capacity, when in charge of unskilled labor, and space permits;

For any capacity, regardless of supervision, when water is variable in character, such as river waters;

For plants where the consumption is variable or considerable elasticity is desired.

The latter type of plant is more generally used in industrial works, where the conditions are more apt to favor it.

The reasons for these general recommendations are as follows:

In the installation of small plants the cost is less than for the continuous process. The attendance is of a very ordinary character, which renders necessary the greatest simplicity, both of construction

Mr. Leopold. and operation. The various regulating parts in a continuous plant are much more liable to derangement in a small plant than in a large one. For any capacity, in the hands of unskilled labor, the simplicity in construction and operating directions insures better results. For any capacity, with variable water, regardless of character of supervision, because it is subject to immediate change in treatment to suit the varied character of water. The continuous plant may be constructed to vary chemical feeds in reasonably accurate proportions with the varying consumption, but a variation in character can only be controlled by a hand regulation of the chemical feeds, because a correction by this means can only be effective after a complete displacement of all water contained in the apparatus. It can readily be seen that it must be several hours, even in theory, while in practice it will be longer by from 25 to 50%, provided the change is made at once accurately and correctly. On the other hand, the intermittent plant is capable of immediate correction, as each tank is treated individually to meet its requirements with absolute quantities of reagents.

Where elasticity is required, as in cases where the maximum consumption is confined to a few hours, the continuous plant must be installed to furnish the hourly maximum. The intermittent may be based on the average or slightly above, as the one tank of settled water always in reserve will supply through the peak of the load without the reduction of time necessary for correct results.

The conditions under which the continuous plant has the advantage are as follows:

Where space is limited;

Where the plant must be located at a point inaccessible to the established steam plant, or a second pumpage is a consideration;

Where the water to be purified is constant in character, as many well waters;

When the water to be purified can only be supplied at about the rate of purification;

Where the plant is in direct charge of an operator with proper technical as well as chemical knowledge.

The reasons are as follows:

There are many situations where, owing to lack of space, it is either impossible or very costly to install an intermittent plant of any capacity. Under these conditions, irrespective of character of water, attendant, etc., it is obviously the only thing to do.

When located in an isolated place that would necessitate an independent power-plant, repumpage or constant attendance, it becomes only a question of these items overbalancing any advantage due to the character of water and uniformity of results.

When the water is uniform in character, the plant properly con-

structed and of sufficient size so that the regulating parts are operative, instead of merely traps, and the operator of ordinary intelligence, the continuous system has all the advantages in its favor. Mr. Leopold.

Where the water supply only equals the consumption, the advantages are usually with this form of apparatus. Under these conditions, except in very large plants, there would be required with the intermittent plant an increased number of tanks, adding to the cost of installation, labor and time of attendance; in most cases probably overbalancing any natural advantage in its favor due to character of water in other conditions.

Where the plant is to be under the direct supervision of a chemist, there are many cases where this type of plant would be preferable, otherwise the writer would recommend the intermittent. This must be decided largely by a knowledge of individual conditions.

The peculiar conditions of railway water station work, embracing as it does many of the above conditions, render the continuous type of plant more particularly adapted to railway purposes than the intermittent type, and it is in this work that we see the greatest number of them, many of them accomplishing the best results possible.

As far as originality is concerned, there is very little in any of the designs now on the market, all being more or less a re-arrangement of details of other apparatus. There is some originality, however, in the methods of some designers in attempting to change chemical laws in order to secure results, and their failure has from time to time cast a cloud over the process in the minds of unsatisfied purchasers and their friends. These laws are impartial. Under them no more can be accomplished by one than another, with the conditions right for their operation. Fortunately, builders of water-softening plants are rapidly learning this. Details of appearance, arrangement or construction may be made to suit individual taste or requirement, but the same general conditions must be adhered to by all who would secure successful results.

Fortunately, experience is still the best teacher and there are fewer reconstructed plants to-day than there were a few years ago. There is also a growing appreciation of the value of water softening not only among engineers, but among steam users in general.

In conclusion, the writer desires to say that in his opinion Mr. Handy's paper is a very valuable addition to the publications on this subject, and one that should be in the hands of every engineer, as it gathers the data from widely scattered sources into an intelligent, condensed form.

HANS REISERT, C. E., Cologne, Germany. (By letter.)—A careful study of Mr. Handy's treatment of the existing condition of the art of water purification makes it evident that the author has treated Mr. Reisert

Mr. Reisert. the different systems from a commendably broad and general point of view. His paper shows that he is well informed as to the chemical side of his subject, although he does not seem to be as fully cognizant of the present condition of the art from its engineering side. For example, in discussing the continuous method of water purification to which he correctly gives the preference on the whole, as compared with other methods, he emphasizes, in several places in his article, the necessity and effect of stirring apparatus, apparatus for producing sedimentation and so forth.

As a matter of fact, however, all these arrangements are entirely unnecessary and have long since been supplanted by much simpler constructions. The problem that confronts the engineer, *i. e.,* to produce the most perfect operation of a machine, and yet use the simplest construction, has been solved in a much better way, in the art of water purification, than the author seems to think. The simplicity of the Reisert apparatus is an evidence of its strength. The few and simple parts of this system operate with greater reliability than the complicated machinery which Mr. Handy regards as more highly perfected.

In the following discussion, the proof of the proposition will be adduced that the most perfect apparatus for purifying water is one in which all the movable parts, such as stirrers, tipping-buckets, and the like, have been entirely done away with.

In most cases, also, the arrangement of special devices for producing sedimentation in the reaction chamber and in the reservoir for purified water are unnecessary and superfluous. They complicate the construction and make it more expensive and make its operation uncertain. Owing to the deposit of sludge which is continually taking place, on such devices, they diminish the amount of useful space that is necessary for the various reactions, are of no use whatsoever, and are often harmful in that they cloud the partly clarified water. Of course, if an imperfect filter bed, consisting of wood shavings, has to be used, then one can readily understand the anxiety of the designer to separate the sludge, if possible, by a decanting operation before filtering. A virtue is made of necessity, but the conclusions drawn from such imperfect operation are faulty.

Mr. Handy's paper does not lay sufficient stress, in the opinion of the writer, on the differences that exist in various raw waters in regard to their physical behavior after the softening operation. Some raw waters which are of about the same degree of hardness as others nevertheless act in an entirely different manner with reference to the speed with which the precipitated sludge settles in the clarification step of the process. For this and other reasons it is a mistake to attempt to formulate a hard and fast rule which is to apply to all kinds of water purification. The installation of a ser-

viceable apparatus for purifying water must always be preceded by an accurate determination of the chemical and physical condition and contents of the water to be purified under the most diverse local conditions, and the engineer and manufacturer of water-purifying apparatus, who is the most certain to guarantee and achieve results, is the one who has had the most experience with different waters. Mr. Reisert.

Mr. Handy seems to have gathered material relating to the softening of water for locomotives and for steam boiler use. Just as important as these uses are those relating to the proper preparation of water for industrial purposes in dye works, bleaching factories, cloth factories and particularly in paper factories.

The following brief discussion of certain parts of Mr. Handy's paper is arranged in the same order as that of the paper.

The general propositions in the beginning of the paper give rise to no adverse criticism. The classification of the different methods of purifying water is very comprehensive. On page 8 under the heading "Limitations of Direct Soda-Ash Treatment," it is also to be noted that the stays of the boiler are injured by the direct introduction and action of soda. On page 17, in discussing the installations of the Solvay Process Company's system of purifying water, the following should be added: "The company has in the meantime abandoned the system described on page 17 and has adopted the Reisert system (see page 20)." Following the installation and successful operation of a Reisert apparatus at Syracuse, having an hourly capacity of 8 000 gal., the company referred to, as well as the Semet Solvay Company, is going ahead with the building of Reisert apparatus in their various plants and has already begun the construction of four more installations of this system in their factories at Milwaukee, etc., each of about 6 000 gal. hourly capacity and one with an hourly capacity of 1 500 gal.

On page 19, the following explanation should be added to that part of the paper which treats of the changes which the Industrial Water Company has made in the Desrumeaux apparatus.

This shows that in actual practice the imperfect operation of the interior arrangement of the Desrumeaux settling tank has been made manifest, as well as the utterly insufficient size of the small line saturator which Desrumeaux formerly used. As a matter of fact, it was at one time considered possible to assist the action of a comparatively small-sized lime saturator by means of a stirring apparatus built into the saturator. In practical operation it was demonstrated, however, that the interior capacity of the lime saturator must be made sufficiently large if a clear saturated lime-water is to be produced.

On page 20, the author is in error in ascribing an entirely insufficient value to the conical shape of the lime saturator. Owing

Mr. Reisert. to the conical shape of the lime saturator the available space is utilized in the most complete manner possible, and all injurious and waste space is avoided. All equivalent forms, such as, for example, a cylindrical reservoir with a conical bottom, are likewise included in the United States patent to Dervaux. A cylindrical or prismatic vessel with a flat bottom would not produce a satisfactory result because in such a reservoir some undissolved lime would collect in the bottom of the cylindrical vessel. That the conical shape of the vessel is the best is shown by experience with cylindrical vessels in which the water to be saturated with lime is introduced through the bottom of the vessel. In such cases a conical funnel, widening toward the top, is formed in the mass of lime. In addition to the method, as described, of supplying soda by means of a siphon which rises and falls with the water level in the main reservoir, another means of supplying soda that is also used in some of the Reisert apparatus, should be mentioned. This consists of an ordinary stand-pipe of suitable diameter to which the soda solution is supplied daily after the soda has been dissolved in hot water. By means of an accurately adjusted micrometer valve, raw water is fed in at the top of the stand-pipe, which displaces the soda solution beneath. It has been found that the raw water will not mix with the soda solution, the latter being of greater specific gravity. The soda solution is thus entirely displaced by the inflowing raw water. Before refilling the stand-pipe with the soda solution, the raw water must be run off.

The author's views as to the apparatus for mixing the raw water with chemicals and as to the interior arrangement of the reaction reservoir in the Reisert apparatus must be strenuously combatted. The Reisert apparatus produces the best mixing that can possibly be obtained. The following comment is to be made on this part of the author's paper.

Perfect mixing is produced in the apparatus by energetically feeding the soda and lime solutions to the stream of raw water in a direction opposite to that of the flow of the raw water. The mixture resulting from this mingling of raw water, lime and soda solutions flows down a vertical pipe. By reason of this compulsory feeding of the mixture a thoroughly intimate mixture is produced that cannot possibly be made more perfect. With this apparatus, all devices have been avoided in the interior of the reaction chamber in order that the heavier sludge may drop to the bottom of the vessel without accumulating on interior parts. This desirable precipitation to the bottom of the vessel will naturally be prevented by any contraction of the cross-section of the chamber, produced, for example, by interior blades or other constructions. In the gravel filter which forms part of the apparatus complete clarification is assured. This avoids

the necessity of and renders superfluous every arrangement and apparatus for producing decantation in the clarifier. Mr. Reisert.

The author considers the automatic filter which is provided at the upper part of the reaction chamber ingenious. A more detailed description of the filter may well be in order.

As soon as the material of the filter bed, which need never be renewed, becomes stopped up by the sludge, the water level rises in the filter until it automatically starts a siphon consisting of three concentric pipes. This produces a strong suction and draws air through the filter bed from the bottom to the top, by means of which operation the particles of sludge are torn from the filter bed. After the suction has worked for a time, the contents of a suitable reservoir flows through the filter bed and so frees the bed from all sludge. The siphon stops as soon as the reservoir, just referred to, is empty and the filter bed, completely cleansed, falls back into its original position. The operation is repeated as soon as the filter bed has once more become clogged. When operated under uniform conditions the filter works automatically and with such regularity that the washouts of the filter bed occur at equal intervals of time. The apparatus of 8 000 gal. capacity installed at Syracuse is also provided with one of these filters. In Europe many installations operate successfully with these filters. Among others, the well-known firm of Fraser & Chalmers is running a water-purifying apparatus at their works in Erith (Kent) with a filter which works like clockwork.

The most usual type of the Reisert system, of which a number are in operation in America, consists of the conical lime saturator, described above, which operates continuously, of a distributing apparatus, a reaction chamber entirely devoid of any special apparatus for producing precipitation and of a Reisert filter with air-suction and washing apparatus. With this apparatus, air is blown into the filter bed by means of an injector and, at the same time, water is fed through the filter bed from the bottom to the top, in which operation all the particles of sludge are torn free from the filter bed and escape through a waste-pipe. This enforced washing out of the filter bed takes place once daily and requires only a few minutes (2 to 5 minutes) in order to produce a complete cleansing of the filter bed and to guarantee a filtration of the softened water, making it as clear as crystal.

At this point the following remarks will be of interest. Although not yet introduced into the United States, a new process for purifying water should be mentioned. This process consists essentially in using barium carbonate for precipitating calcium sulphate. In this process, the scale-forming salts are not partly changed into soluble salts (sodium sulphate), which, by evaporation in the boiler, turn

Mr. Reisert. into a more and more concentrated lye, as in the soda and lime in soda process, but the salts are entirely removed. From this the following advantages result:

There is no leakage of sodium sulphate at the stays and thin spots of the boiler, and, therefore, great security and strength of the stays and also increased safety in the operation of the boiler; no foaming of the water in the boiler; no "working water" of locomotives and of steam engines; no incrustation of feed-water heaters, feed-water pipes and injectors. Although barium carbonate is insoluble, nevertheless, when used in the manner prescribed by the Reisert process, it has the property of uniting with dissolved salts. Free as well as combined sulphuric acid is taken up by barium carbonate, and calcium sulphate, which is the principal agent that forms boiler scale, is transformed into simple carbonate of calcium and barium sulphate, both insoluble salts which are precipitated in the form of sludge. An additional advantage of the process is that the carbonate of barium does not have to be supplied in a carefully predetermined amount— but that it may be dumped once for all into the water-purifying apparatus for a long run, for no more of the material can be used than the amount necessary to remove the sulphuric acid from the water, because the salt is insoluble in water. If the water also contains calcium carbonate, this will be precipitated as a simple carbonate of calcium in the sludge, by acting upon it by means of the saturated solution of the lime-water. The latter is supplied by the lime saturator in a uniform and continuous solution. Water containing corroding chemicals, as, for example, magnesium chloride, is treated in a suitable manner, so that the corrosion can be avoided entirely. The clarification of the water occurs in a Reisert gravel filter. Any oil or iron contained in the water is entirely removed by the filter. The new process is especially advantageous for locomotive boilers because it prevents the locomotive from working water, and because it is unnecessary to discharge or blow out the boiler water of the locomotive by reason of the concentration of the salts in the water.

That which the author has elsewhere called an advantage of the Kennicott apparatus, *i. e.*, its compact arrangement, is peculiarly an advantage of the Reisert apparatus in contradistinction to any other system. On page 24 of Mr. Handy's paper, the use of soft water is recommended for the preparation of lime-water. In this matter, the author seems to be mistaken. When such water is used for producing saturated lime-water, a non-uniform saturation is produced by the formation of varying amounts of $Na(OH)_2$. No stress is laid in the paper of Mr. Handy, which has evidently been prepared with painstaking care, upon the advantage of using purified water for factories in all branches of the textile and paper art, etc. The

large silk factories in Paterson, near New York City, and other Mr. Reisert factories in the textile industry are using purified water with great advantage for washing and for other purposes. The preparation of this water must be carried on with care so that it is of the right degree of softness, but is not too alkaline. Complete clarification is essential. This can only be accomplished by means of a reliable filter acting in co-operation with the water-softening apparatus. Ordinary wood-shaving filters are too unreliable for this purpose, because these may permit the passage of sludge which will dirty the purified water in the collecting reservoir. This entails great damage to the manufacturer. Apparatus making use of the above-described gravel filters of the Reisert Company have shown themselves well adapted for these uses, for with them it is impossible that sludge should pass through and reach the reservoir for purified water.

In factories in the textile art and in paper factories large amounts of clear water are often used. It often happens that the water which is to be used must be taken from streams and cannot be filtered clear in a simple way. In such cases, installations of the Reisert Company have achieved excellent results. In these installations, a large, walled-in reservoir has been built to receive the water to be filtered. The water is uniformly treated with a thin solution of aluminum sulphate or any other coagulant. Experience has taught that in such reservoirs 2 to 3 hours' reaction time must be given to the water.

G. M. Campbell, Esq.,* Pittsburg, Pa. (By letter.)—The paper Mr. Campbell contains much valuable information, coming as it does from one so conversant with the art of water treatment as Mr. Handy. The various features of the different machines and the larger benefits to be derived from water softening are well brought out. These points will be but lightly touched on in this discussion, and attention chiefly directed to the last paragraph of the paper, "Chemical Control of Softening Plants," and also to the general administrative methods of handling the softeners on the Pittsburgh and Lake Erie Railroad, several references to which company and the results obtained are found in the paper under discussion.

There are in use on this road ten Kennicott softeners, eight using water from six different rivers, one using well water and the last using water from a shallow pond. Some of the waters are high in carbonates, others high in sulphates, some are as high in free sulphuric acid as 25 gr. to the gallon. These waters, which vary within very wide limits, are being treated with entire success. The hardness is reduced to from 2½ to 4 gr. of calcium carbonate per gal., which will not form any scale whatever. Some of the waters when treated contain a relatively large amount of soluble salts, chiefly

* Electrical Engineer, Pittsburgh & Lake Erie R. R. Co.

. Campbell. sodium sulphate. These salts, after concentration, sooner or later, cause the boiler to prime. Just what the concentration has to be before trouble commences has not been determined. Some engineers report priming when the water shows on test only 20 to 30 gr. of soluble salts to the gallon, others report locomotive satisfactory when the water contains 100 to 150 gr. to the gallon. Such water is always "solid," there is never any doubt as to its height in the boiler. As to stationary boilers, experience in connection with two power-houses, where water-tube boilers are used, seems to indicate that so long as the water is free from suspended matter or oil, no priming occurs. Water has been taken from these boilers showing as high as 350 gr. to the gallon of soluble salts, and in some of these cases the sodium carbonate or sodium hydrate reached 150 gr. to the gallon.

In the dry summer months, before the water softeners were installed, sometimes as high as 50% of the locomotives would be out of service from causes directly attributable to bad water. Since the installation of the softeners, troubles from acid waters have ceased entirely, nor is any scale formed in boilers using treated water only. There is some trouble from leaking, especially in the boilers with wide firebox, but this is due to other causes than to the water. There is very little trouble from priming or foaming, owing to the frequent changes of water. This is accomplished without any racking of the boiler, by means of the Raymer hot-water washing-out appliance referred to in the paper. Passenger engines are washed out every 20 days, freight, every 45 days; water is changed in the former every 2 to 5 days, and in the latter every 5 to 12 days.

At each of the water-softening points, the pumper looks after the softener also; many of the pumpers are mere boys. Each pumper makes his own tests of the raw water and puts into the machine the amount of soda and lime called for by certain tables given him. The accuracy and reliability of the tests made by the pumpers is remarkable. Each man keeps a complete record of all tests and charges on a weekly report sheet. He also collects on three days each week samples of raw water and samples of treated water; these are forwarded to the laboratory and tested by the head inspector or chemist. The inspector visits, at irregular intervals, the various softeners, to note conditions. The softener attendant is not held responsible for the quality of the water after treatment; he is held responsible only for the putting in of certain charges of lime and soda and for keeping his machine in as good order mechanically as his pump.

The raw water is tested for hardness, alkalinity and acidity (when the water is alkaline, the acidity to phenolphthalein as indicator is due to the presence of free carbonic acid). The soda

charge depends on the difference between the hardness and the alkalinity, the lime charge depends on the sum of the alkalinity and the acidity. All readings are, on this road, expressed in parts of calcium carbonate per 100 000, each part being called 1°. About 0.1 lb. of soda are required for each 1° of hardness less alkalinity, and about 0.065 lb. of lime for each 1° of alkalinity plus acidity. Mr Campbell

In order to determine whether the treatment is correct, the treated water is tested for hardness, alkalinity and causticity, a word coined by McGill,* but here used to mean the reading obtained by adding fiftieth normal solution of sulphuric acid to 200 cu. cm. of water with phenolphthalein as indicator. These three simple tests give complete and accurate information as far as scale forming is concerned. The treatment is full and complete when hardness = alkalinity = causticity = 6 or less when expressed in parts per 100 000, *i. e.*, about $3\frac{1}{2}$ gr. to the gallon. It is, of course, not always possible to obtain this exact result. For locomotive work the alkalinity should be, if not equal to the hardness, slightly below it. For stationary work the alkalinity should be one or two points in excess of the hardness. In no case should the causticity exceed the alkalinity.

The raw water varies considerably in hardness, etc., the treated water only slightly. For the week ending September 3d, 1904, when conditions were about average, the amount of water treated was 10 686 854 gal.; the average hardness of the raw water was 24.3; average hardness of treated water, 6.02; alkalinity, 5.96; and causticity, 5.98, all figures being in parts of calcium carbonate per 100 000. The chemicals used were 13 727 lb. of soda and 16 043 lb. of lime, which at 0.85 cent and 0.3 cent per lb., respectively, would amount to about $165.00 for the week, or a cost of 1.545 cents per 1 000 gal. treated.

In order to treat any water satisfactorily, simple tests on the ground are absolutely essential. Treatment based on occasional complete chemical analysis by an expert chemist is not close enough for variable waters or even for fairly constant well waters, if refinement, such as is aimed at on this road, is desired. The alkalinity, acidity and causticity tests are, of course, definite chemical reactions and are, therefore, accurate within the errors of observation as practiced on the ground, but, unfortunately, the hardness, as determined by the soap test, is considered, by the public in general and the chemical fraternity in particular, as a mere approximation. Exhaustive experiments have been made in the Pittsburgh and Lake Erie Laboratory, and as a basis of comparison a large number of duplicate samples were analyzed by the Pittsburgh Testing Laboratory. The results prove conclusively that the soap test is a thor-

* *Journal*, Society of Chemical Industry, 1904.

Campbell. oughly reliable and accurate method of determining hardness, in so far as it bears on water treatment.

The results for water softening on this road are satisfactory beyond question, and much of the credit is due to Mr. Handy, who has acted as Consulting Chemist.

Mr. Miller. FRED J. MILLER, ESQ., New York City.—Attention is called to a statement made on page 5 of the paper to the effect that:

"Schemes for heating boiler feed-water under pressure before passing to the boiler have never passed the experimental stage, owing chiefly to the imperfect precipitation of scale-forming substances by short exposure."

The speaker believes this statement to be too broad to be accurate and calls attention to the Hoppes system of feed-water purification, which is by heating the boiler feed-water under pressure before passing to the boiler, and which for years has been far past the experimental stage and entirely successful in a large number of cases of stationary boiler plants. In St. Louis alone, in a single establishment, namely, the St. Louis Transit Company, 33 000 h. p. of its boilers have attached to them these live-steam feed-water purifiers—a fact which seems to indicate that this method of purifying boiler water has passed far beyond the experimental stage. This system consists of a closed heater, capable of sustaining boiler pressure, placed above the level of the boiler and containing sheet steel pans having semi-circular or cylindrical bottoms, placed one above the other, and so arranged that the feed-water fed into the uppermost pan flowed in a thin stream over the edges of this pan, adhered to the under side of it until the middle of the pan was reached, then dropped from that pan to the next lower one, and so on through the purifier, being, during this action, exposed to steam at full boiler pressure. Practically no scale is deposited on the inside of these pans, but it is all deposited on the outside of them, probably upon the stallactite principle. There have been cases in which water so thoroughly impregnated with lime as to be with difficulty used in boilers at all had by the use of these purifiers been rendered so clear of scale-forming materials that it was actually too pure for boiler use, and would, therefore, set up oxidation in the same manner as has been found to take place by the use of rain water or distilled water. In some cases the action of the purifiers has been suspended for a short time, at regular intervals, in order to maintain a thin coating of scale to protect the interior surfaces of the boilers from oxidation and pitting.

The speaker did not know the exact cost of water purification by this method, but believed the cost to be small where the heater is protected from radiation, and where the water passes immediately

to the boiler before the heat imparted to it by the steam becomes dissipated. He never heard of its application to locomotive practice, and believed it to be inapplicable to that service. He merely called attention to the statement quoted as being too broad in view of the facts which he cited. Mr. Miller

JAMES O. HANDY, ESQ., Pittsburgh, Pa. (By letter.)—In closing the discussion the writer begs to express his indebtedness to those who have contributed to it. Mr. Handy.

Mr. Booth's reasons for the use of a half-hour stirring period and a lime-water tank of large size are certainly worth considering. If preliminary experiments with a water and the available purifying agents indicate the necessity of longer agitation, or other changes from the usual design of plant, or method of operating, they should be made.

Mr. McGill is evidently unfamiliar with the design and operation of continuous softeners of the "Kennicott," or "Industrial" class. Otherwise he would not question the fact that the continuous system of water softening which they typify is the best product of engineering and chemical skill applied to the purification of hard water. It is possible to yield to the "fascination" which Mr. McGill says "continuous" softening systems have, and feel perfectly sure that no necessary precaution has been neglected or "chemical principle ignored." The time allowed for chemical reaction is usually the same in continuous and intermittent systems, but the tank area required by the intermittent system is twice as great, the ground space greater and operation more costly.

Replying to Mr. Peabody, the writer does not know of cases of foaming of stationary boilers due to sodium sulphate. Mr. Campbell gives data on this point showing that the concentration may go as high as 350 gr. of soluble salts per gal. without causing foaming. Sodium carbonate for several reasons does cause foaming when concentrated by evaporation in boiler water. By its action on organic matter, or by the precipitation of solid matters, it adds to its own inherent disturbing properties.

In Raymer's system nearly all the B. t. u. in the foul water are utilized to heat the new water.

It will hardly be necessary to refute Mr. Leopold's statement that the use of Goldsmith's system of softening is the only way to insure absolute accuracy of treatment. To weigh out a certain quantity of commercial lime which may be quite variable in quality and to apply it, after slaking, to a certain volume of water is not as safe or as accurate a method as to apply saturated lime-water proportionately over a weir or through a slotted pipe. Mr. Campbell's remarks indicate that unskilled labor successfully operates continuous plants. The matter of variable consumption is cared for by a

Mr. Handy. storage tank and the softener operated at the rate found to be necessary.

Mr. Reisert considers the writer ignorant of the engineering features of water softening because his design of softener is not endorsed. It is impossible to approve a design in which less than two hours are allowed for softening and no provision made to ensure steady passage of the water, undisturbed by eddies. Opportunities for studying the working of a Reisert apparatus have been frequent in the past two years, during which some half dozen unsatisfactory trial runs have been made by the contractors for the purpose of convincing the Pittsburgh and Lake Erie Railroad Company that they should accept one of these plants. In the meantime, ten plants of another make have been installed and are in satisfactory operation.

For the benefit of those who think purifiers of the "preliminary heater" type are a good thing, it must be said that such purification is incomplete even at best, and that the scale in the heater interferes with the proper heating of feed-water which is absolutely essential.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

INTERNATIONAL ENGINEERING CONGRESS.

1904.

FORTIFICATIONS.

Congress Paper No. 2.

By George W. Goethals, Maj., Corps of Engrs., U. S. A.

Note.—Figures and Tables in the text are numbered consecutively through the papers and discussion on each subject.

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FORTIFICATIONS.

By George W. Goethals, Maj., Corps of Engrs., U. S. A.

The art of war is usually divided by military writers into four principal branches: strategy, tactics, fortification and logistics, and while they are more or less dependent on each other, the first three are very intimately allied. Strategy determines the location of a position to be occupied by troops in the furtherance of a campaign: tactics disposes of the troops on the position selected, while fortification improves the natural features of the position so as to increase the chances of success. Fortification has, therefore, been defined as the art of increasing, by engineering devices, the fighting power of troops occupying a position for receiving or making an attack.

If the works are constructed by an army in the field, or during the progress of the war, making use of materials at hand, they are temporary in character, and belong to what is termed field fortifications. If they are designed and built during time of peace with all the resources of the State available, they constitute permanent fortifications. The former may be hastily constructed, as in the case of intrenchments, or simple trenches thrown up by troops in the presence of the enemy; they may be of stronger profile designed to protect some depot, city or important strategic point against the ordinary attack of a field army, or they may be siege works under-

taken with a view to reducing fortifications erected by an enemy for the protection of some position. However they may differ in construction, the object of each is the same, namely, to so modify the natural features of a position as to increase the destructive effect of the fire of troops occupying a fortification, at the same time decreasing the effect of the enemy's fire.

With the exception of the trenches thrown up by American troops in the campaigns of Santiago and in the Philippines, there has been no necessity to resort to works of the field type within the last decade in the United States.

Permanent fortification embraces the works designed and constructed to retain possession of important strategic positions in a State, such as manufacturing and railroad centers, political capitals, important mountain passes, naval and commercial harbors, etc. They are placed at important points along or in the vicinity of the frontiers to prevent invasions and to give bases for offensive operations against an enemy, and along the seacoast to prevent the capture, or bombardment, of commercial ports by an enemy's fleet, and to form bases from which the friendly navy can operate, or where they can obtain shelter in case of defeat; the former constitute the permanent land defenses, the latter the seacoast defenses. Differing in their object and their armament to suit the difference in the character of the attack, the principles governing their location and construction are the same.

In a strategic sense the seacoast has been likened to a mountain chain along the frontier, the bays and mouths of rivers being analagous to the natural mountain passes, and as these passes form the highways between adjacent countries, and cities are found on either side of the divide, so, on bays and at the mouths of navigable rivers, commercial and manufacturing centers naturally spring up. Seacoast fortifications, therefore, correspond to the land fortresses along or near the frontier.

The points along the seacoast, which should be fortified, must depend upon the object of a naval attack, and this may be to gain possession:

1.—Of important commercial ports; they are, from the natural condition of things, situated on bays and mouths of rivers having ample depths for the deeper draft commercial vessels, therefore, suit-

able bases, not only for a friendly, but also for an enemy's navy. They are, as a rule, important railroad centers and, from this combination of land and water transportation, centers of supply. The coaling stations, navy yards and naval stations are generally at or near them. They may also be depots of naval and military stores, in which case their seizure becomes all the more important to an enemy, and more disastrous to the country attacked.

2.—Of vessels engaged in foreign trade; while at sea this duty would devolve upon the navy, but when near the coast they should find fortified harbors within which they can obtain secure refuge.

3.—Most particularly applicable to the United States because of the great extent of coast line and the great number of ships concerned, of vessels engaged in the coasting trade; this protection in a measure might also devolve upon the navy, but cruisers are better utilized looking after the destruction of an enemy's commerce than in an almost hopeless attempt to protect the coasting trade.

Seacoast defense "must close all important harbors against an enemy and secure them to our commercial and military marine. Second, must deprive an enemy of all strong positions, where, protected by naval superiority, he might fix permanent quarters on our territory, maintain himself during the war and keep the whole frontier in perpetual alarm. Third, must cover the great cities from attack. Fourth, must prevent, as far as practicable, the great avenues of interior navigation from being blockaded at their entrance into the ocean. Fifth, must cover the coastwise and interior navigation by closing the harbors and the several inlets from the sea which intersect the lines of communication, and thereby further aid the navy in protecting the navigation of the country. And sixth, they must protect the great naval establishments."*

As a rule, the number of places that must be defended along the seacoast is greater than along the land frontier, for hostile vessels are likely to enter any port where there is shipping worth destroying, and to subject any seacoast town to bombardment or indemnity, so that every commercial port along the seacoast should be protected, whether or not they are of military and naval value. On the other hand, the allotment of funds for seacoast defenses is limited, and permits fortifying only those of the greater strategic importance.

The ports or harbors to be fortified are usually determined by a

* Report by Bernard and Totten. 1826.

board consisting of officers of the army and navy, and, in the United States, the selection has been made by the "Endicott Board," which consisted of the Secretary of War, two officers of Engineers, two of Ordnance of the Army, two officers of the Navy, and two civilians, appointed by the President in compliance with an Act of Congress, "to examine and report at what ports, fortifications or other defenses are most urgently required, the character and kind of the defenses best adapted for each, with reference to armament" and "the utilization of torpedoes, mines or other defensive appliances." The Board, in its report, designated twenty-seven ports of the United States at which fortifications are most urgently required, one of this number being "the Lake ports;" the report states further that, in limiting the list to the names given, "the Board does not wish it to be understood that there are no other places along the coast which are of the importance necessary to justify measures of defense." To the ports mentioned in the list, a few others were added subsequently by reason of very rapid commercial growth, or for the protection of naval bases undertaken since the rendition of the report. In this way the location of the seacoast defenses of the United States has been determined.

In days when vessels were propelled by sail, it was necessary for ships to remain practically stationary in order to attack coast defenses, and, therefore, until within a comparatively short time the coast defenses of the United States consisted of stone forts with tier upon tier of casemated guns, each fort provided with proper defenses against land attack. With the introduction of steam, however, the conditions have been materially modified. It is no longer necessary for ships to anchor, and in order to prevent the ships from concentrating their fire on the coast batteries, it has become necessary to scatter the guns, making a number of independent batteries in lieu of concentrating all the guns in one position; this is, however, an advantage to the defense. The introduction of steam and the improvement in machinery has developed such speed that it becomes possible for ships to run by shore guns, thereby necessitating the use of some means to hold them in check, and there has resulted the system of submarine mines. In some localities, due to the great depths or strong currents, submarine mines are difficult to maintain, and some form of torpedo operated

or projected from the shore must be used. To prevent tampering with the mines, guns ashore are needed to protect the mine fields, and search-lights must illuminate the fields so as to disclose attempts which may be made at night for the removal or destruction of the mines; and torpedo and submarine boats must be provided to operate against similar craft utilized by the enemy for opening a route through the mines.

There are harbors with entrances so wide, or soil too poor for proper foundations, that they cannot be effectively closed by land batteries and submarine mines, and, in such case, the land batteries must be supplemented by floating defenses—not naval vessels in the proper sense of the term, but so-called "harbor-defense vessels," speed and sea-worthiness being sacrificed to secure heavy armament well protected by armor.

Wars of recent years have shown the impossibility of reducing fortifications, if properly defended, by ships' fire, and, when fortified ports have fallen, such a result has been secured by land operations assisted by a blockade; the necessity for a proper land defense in connection with coast defense is, therefore, clear.

To sum up, an effective system of coast defense must consist of land batteries, with their protecting guns and searchlights, submarine mines, torpedoes, torpedo and submarine boats, floating defenses and proper land defenses.

To prevent distant bombardment by a fleet, and also to prevent the forcing of a passage or a running by the defenses, high-power guns are required for disabling or silencing battle-ships and cruisers at long ranges, and, in addition to the mines, smaller guns of the rapid-fire type are needed for similar purposes, and to repel torpedo-boat attacks at the closer ranges.

Mines are essentially obstacles, and, to accomplish their object, must succeed in holding the enemy in the zone of greatest effective fire; they must allow safe passage of the vessels of the defense, but must be instantly dangerous to the enemy's ships.

The types of guns to be used in coast defense have received very careful consideration. The target for the artillery is either the side or the deck of a ship. To reach the former, direct fire is necessary; to reach the latter, curved fire must be used. In determining the caliber, as well as the number, of the guns for the defense, Gen-

eral H. L. Abbot, Corps of Engineers, has laid down the principle that "for a port of first-class importance, the armament should never be allowed to fall below that of the enemy in caliber, and that in number of guns we should rarely mount less than half of what can be deployed against the works in line of battle."

A study of the battle-ships, their armament and thickness of armor, the necessity of forcing them to begin the battle at extreme ranges by requiring the high-power guns to have sufficient energy to penetrate the armor belt, at least at a two-mile range, has led to the adoption of the 8-in., 10-in. and 12-in. breech-loading rifles for use of the defense. One 16-in. gun has been constructed for use in New York Harbor, but it is questionable whether any more of this type will be made, as improvements in the manufacture of heavy ordnance have resulted in securing a 12-in. gun which, for effectiveness, is not surpassed by naval guns of even higher caliber.

Curved fire is necessary to reach the most vulnerable part—the ship's deck, and howitzers and mortars secure this result. From experience with mortars during the war of 1861-65, and from experiments made abroad, the United States has adopted the 12-in. breech-loading rifled mortar. They are comparatively cheap, are easily concealed from view and direct fire from the ships, and are arranged in pits containing at least four; as they are fired simultaneously, the chances of hitting the target are very great. Considerable prejudice was incited a few years ago against the extensive use of the mortar in the defenses, and, in consequence, extensive tests were made both on fixed and moving targets at ranges varying up to 12 000 yards; the results were much better than even the strongest advocates of the mortar hoped for, and this portion of the armament is now regarded by all as one of the most important.

For fighting at closer ranges, to assist in frustrating an attempt to force a passage, to cope with the rapid-fire armament aboard ship, to repel landing parties, and protect the mine fields, guns of smaller type are necessary. In the United States service these consist of the 6-pounder, the 15-pounder, the 5-in. and the 6-in. rapid-fire guns. The characteristics of these various guns are given in Table 1.

To determine the position of the batteries, circles are drawn with radii of one, two and three miles, respectively, on a chart of the

harbor to be fortified, various available sites being used as centers. From this chart, and from the circles described thereon, the possible extent of front which an attacking fleet can assume, the class of ships which available depths will permit, and the armament are determined. The disposition of guns, etc., is then determined by the same tactical principles as apply to the placing of a land force in position. In this case, however, the ships being confined to navigable channels, any possible flank attacks can be foreseen and guarded against; on the other hand, no change can be made in extending, or changing front, as in the case of land forces. The following considerations must, therefore, have weight in making selection of the sites to be occupied:

TABLE 1.—HEAVY ARMAMENT.

Caliber.	Charge, in pounds, smokeless powder.	Weight of projectile, in pounds.	Muzzle velocity, in feet per second.	Muzzle energy, in foot-tons.
8-inch......	75	300	2 250	1 052
10-inch......	150	575	2 300	21 090
12-inch......	260	1 000	2 300	36 675
16-inch......	540	2 000	2 300	73 350

RAPID-FIRE ARMAMENT.

Caliber.	Weight of charge, in pounds.	Weight of projectile, in pounds.	Shots per minute.	Muzzle velocity, in feet per second.
6-pounder..	1	6	20	1 800 to 2 500
15-pounder..	3	15	20	1 800 to 2 500
5-inch......	30	50	10	2 300
6-inch......	50	100	6	2 500 to 3 000

MORTARS.

Caliber.	Charge, in pounds.	Weight of projectile, in pounds.	Muzzle velocity, in feet per second.	Muzzle energy, in foot-tons.	Range at which it can penetrate armored decks.
12-inch......	105	800–1 000	1 140	7 206	1 to 7 miles.

The batteries must be placed so as to permit the use of an efficient system of submarine mines located well to the front of the

general line of defense. While, at one time, submarine mines were regarded merely as accessories to the seacoast defense, they must now be considered as one of the most essential features, almost, if not entirely, on a par with the gun in importance.

A front attack on a fortified position is the most difficult and hazardous, and offers the smallest chances of success; the general line of defense should, therefore, be so arranged as to force the fleet to a frontal attack, if the capture of the place warrants the attempt. All side channels, in which ships might take position and secure a flank or reverse fire on any of the batteries, should be closed by obstructions or be properly defended.

Any disposition which necessitates a reduction in the front, which a fleet can assume, will be to the advantage of the defense, and batteries, widely distributed, will prevent concentration of fire on the coast batteries, but will enable the latter to bring a concentrated fire or a very effective cross fire on the ships.

All important anchorages and channels beyond the line of defense, and within effective gun or mortar fire, must be thoroughly covered by the fire of the batteries. In short, there should not be any dead spaces or angles which ships might occupy to the disadvantage of the defense.

As between high sites and low, the former are preferred; for, while not adding materially to the offensive power of the gun, they greatly decrease the effect of the enemy's fire, but since the batteries, as designed, permit a depression of the gun not greater than from 5 to 7° from the horizontal, care must be taken that the dead spaces, resulting from the elevation, are properly covered. Low sites, while permitting ricochet fire, place the guns ashore more nearly on a par with those afloat.

The great advantage of fortification is the cover which is given to the defenders. In the old, stone, casemated forts this was secured to the greatest possible extent, even to closing the embrasures against small-arms fire, shrapnel, etc., when the guns were withdrawn for loading. In the modern system of defenses the attainment of this object has also been sought, and is secured not only by the parapet, but by the use of a suitable mount by which the gun can be loaded under cover.

When Congress first made appropriations for the construction

of the modern coast defenses a suitable gun-carriage had not been devised, and, because of the great importance attached to the effect of mortar fire, the earlier allotments were expended in the construction of mortar batteries, and in the testing of various gun-carriages for the heavier armament to be installed. A careful and analytical study of results secured by experiments with mortars abroad, by General H. L. Abbot, Corps of Engineers, led to the adoption of the present arrangement of the mortar battery. Four mortars are grouped together in a pit and two or four of these pits into a battery. The discussion showed that the best results were attained by four pits located at the corners of a rectangle, and this type was adopted where suitable sites were found, but local conditions and economical considerations have, in most cases, required a variation and the distribution of the pits along a straight line. The magazines, shell rooms, bomb-proofs, etc., are placed in traverses separating the pits and under the parapets. To resist penetration a thickness of parapet equivalent to at least 70 ft. of sand was adopted, and is still practically the standard.

The mortar battery of to-day differs from the earlier construction only in the extra size given to the pit, necessitated by the construction of a more powerful piece, by the increased facilities for loading and by rapidity of fire. Objections were raised to the extensive use which was being made of mortar batteries in coast defense, on the ground of inaccuracy of fire, which led Congress to omit, for a few years, appropriations for their further construction, but after the exhaustive tests already referred to, and which showed conclusively the value of these weapons, recent appropriations again authorize them.

The types of mounts for the guns received very careful investigation and study, and consisted of the lift, the disappearing and the non-disappearing or barbette carriages. In the lift, the gun and platform are raised from the loading to the firing position and lowered to the loading position by suitably designed machinery. The disappearing gun-carriage utilizes the energy of recoil in raising the gun to the firing position. The barbette carriage is the adaptation of the one formerly used to the more powerful gun subsequently adopted.

In the case of the gun-lift, the greatest amount of protection

is given to both cannoneers and the gun. The latter is exposed to direct fire only when in the firing position, making a very small and momentary target for the ships. Its great cost rendered the substitution of some type of disappearing gun-carriage necessary, to the entire exclusion of the gun-lift; but one battery of this latter type has been constructed, and, in this case, hydraulic power is used.

After exhaustive tests the Buffington and Crozier gun-carriage was adopted for the United States service; the recoil raises the counterweight which, in falling, again raises the gun to the firing position. The emplacements are designed so as to give protection to the cannoneers, standing on the rear of the gun platform, from all shot likely to come over the parapet. Barbette mounts, while used, are not so general in their application as they expose the gun at all times and the gunners, while operating it, to direct fire.

Considerable opposition was raised, in 1901, to the use of the disappearing gun-carriage in the defenses of the United States, on the ground that accuracy in fire was not always possible, the greater expense in the cost of the emplacement and of the carriage, and because this type had been practically abandoned by all foreign powers. As a result, Congress, in the Act approved June 6th, 1902, provided that none of the money appropriated by the Act should be expended for disappearing gun-carriages until a thorough test had been made by a disinterested board of officers of high rank and, at least, one mechanical engineer of high standing. A board, consisting of one officer of Engineers, one of Ordnance, three of Artillery of the Army, one officer of the Navy and one civilian mechanical engineer, was appointed, under the Act, to make a thorough and exhaustive test of the barbette and disappearing gun-carriages. This was done by firing, under service conditions, 6-in., 8-in., 10-in. and 12-in. guns, two of each caliber, on disappearing and non-disappearing types.

As the result of these tests the conclusions of the board were as follows:

"The elevation of the site above the sea level does not materially affect the choice of the style of carriage to be mounted.

"With the larger guns, 10 and 12-in calibers, greater rapidity of fire was obtained with the disappearing type. The advantage in time and less fatigue of men are sufficient to justify a decided preference for the disappearing mount. With the 8-in. gun the

rapidity of working by men of equal skill would appear to be substantially the same, though the labor would be greater for the non-disappearing type. With the 6-in. gun the advantage was decidedly in favor of the non-disappearing carriage.

"With the non-disappearing carriage, without shield, substantially every man working the gun, except those on the elevating cranks, is exposed to direct fire, while with the disappearing carriage and direct laying only one is exposed, and with indirect laying no one is exposed for small angles of fall. With the non-disappearing Armstrong carriage with shield the gunner has better protection than he has on any other mount examined by the board.

"The gun on a non-disappearing carriage is exposed at all times, while with the disappearing carriage it is exposed at each fire with direct laying about seven seconds plus part of the few seconds required for aiming. When the gun on a disappearing mount is laid indirectly it is exposed only about seven seconds at each firing. To direct fire the vulnerability of the disappearing carriage is greater than that of the non-disappearing during equal times of exposure, but the time of exposure of the disappearing type is insignificant relative to that of the non-disappearing.

"The disappearing mount is more vulnerable to fragments of shell, concrete, sand, etc., falling inside of the angle of protection by reason of the character and the greater number of working parts.

"Considering the two mounts merely as machines, the disappearing contains more moving parts and appears more susceptible to mechanical derangement, but the extent of this additional complication is measured chiefly by the addition of counterpoise weight, vertical crosshead guides, tripping device, the two gun levers, two additional pairs of trunnions and the additional weight imposed on the base.

"Both carriages contain parts of substantially equal complication in hydraulic cylinders with pistons, piston rods, throttling bars and equalizing pipes, and the rollers on which the carriage moves into battery are not necessarily more complicated in one type than in the other. The traversing mechanism and the gearing and auxiliary mechanism by which the gun is pointed at the proper angle of elevation appear little, if any, more liable to derangement in one type than in the other.

"It is to be noted that the pressure in the recoil cylinders and equalizing pipes of the non-disappearing carriages is greater than in those of the disappearing type.

"Certain experiments made by the board with sand scattered over the chassis rails of both types of carriage showed that the mechanism of the disappearing carriage as now constructed was more susceptible to such obstruction.

"The mechanical differences of the two types of mount are not in themselves sufficient to determine the choice of type.

"Liability to derangement of platform or base is, in the opinion of the board, greater in disappearing mounts. The board, however, has had no opportunity of observing this derangement, but it is of the opinion that it is of remote probability with the latest type of installation, except on sites where the foundation may be unstable.

"*Derangement of aim by failure to go into battery.*—The advantage is materially with the non-disappearing mount; with the latest type of carriage no such failure occurred during the tests of the board.

"As between the non-disappearing carriage without shield and disappearing carriage there does not appear to be sufficient difference in cost of emplacement and mount to materially influence the choice. With the proper shield added to the standard United States non-disappearing mount, it appears that the cost would equal or more probably exceed that of the disappearing.

"*Target practice.*—Under service conditions with full charges of smokeless powder the two types of mount are equally suitable. The non-disappearing is much better adapted to sub-caliber practice and firing with reduced charges.

"A premature discharge of a gun on a disappearing carriage while below the parapet would be far more disastrous to the mount and men serving the gun than would be the case with premature discharge of a gun on a non-disappearing carriage. When a lanyard is used for firing, the act of the disappearing gun in rising into battery increases the liability to premature discharge by the possible fouling of the lanyard, except in the case of the 6-in. gun with short lanyard of the latest firing device.

"*Effect of blast on neighboring gun detachment when firing toward the flank of a battery.*—The interference with the neighboring detachment would be much more serious with the non-disappearing type of carriage."

The detailed plans of batteries are confidential, and, therefore, only a general description is permissible; while, in any particular case, the plans are dependent largely on the particular site which the guns are to occupy, in general, the ruling type consists of a parapet, the interior crest of which is about 20 ft. above the general lay of the land, with the gun platform, approximately, 10 ft. below this crest. Under the parapet proper are the magazine and shell rooms, for the storage of powder and projectiles, with galleries, guard rooms, office rooms, etc., distributed in rear of the magazine and shell rooms, and under the gun platform. The depth

of the platform is determined by the length of the gun and rammer, being equal to the sum of the two, though in contracted sites this desirable depth is not always attainable. The crest in the immediate vicinity of the gun is circular, the length of arc depending upon the field of fire of the gun, and breaks back along a traverse which separates adjacent guns in a battery.

The horizontal and vertical protection now given to the magazines and guns is based on the penetration of the 12-in. B. L. Navy Rifle, firing an 850-pound projectile with an initial velocity of 2 800 ft. Taking the penetration in Krupp armor at 21.7 in. at the muzzle, 16.2 in. at 3 000 yd., and 9.05 in. at 5 miles, with an angle of fall of about 7°, the relative value of protection has been taken at 1 Krupp armor equal to 3 wrought iron, 27.6 Portland concrete, and 82.8 sand. The protection given to the recent types is, therefore, 15 ft. of concrete and 45 ft. of sand for all walls exposed to horizontal fire, and 10 ft. of concrete where exposed to vertical fire.

In 1898, it was thought by certain officials of the War Department that the cost of emplacements could be materially reduced by the substitution of iron rails for the concrete in the parapet. It was proposed to place the rails on end, holding them in position by a footing of concrete, with a slight inclination to the rear, as this arrangement, it was claimed, would give a tendency to deflect the shot should it penetrate to the row of rails. A comparative test of a parapet with concrete and with rails was ordered.

Two sections of experimental parapets were constructed and subjected to the fire of a 10-in. gun; one section was of concrete, 20 ft. in thickness, and earth; the concrete, similar in dimensions and character to that used in the parapets of batteries already constructed, was of Rosendale cement for the body of the mass in the proportions of 1 to 2 to 4, with the upper 3 ft. of Portland cement; the second section was of earth, the interior slope revetted with three rows of 90-lb. steel rails firmly held in place in a footing of concrete. These were surrounded by shields or butts to hold the shot, and back of each section was a screen to register the damage which might be expected from the shot and débris.

In conducting the experiments cast-iron projectiles weighing 575 lb. were used, and an angle of impact of about 20°, correspond-

ing to a range of 9 000 yd. Four shots were fired at the first section of parapet; but one at the second. The first shot struck with a calculated velocity of 1 100 ft. per sec., equal to the final velocity with full charge at 9 000 yd. range. The projectile struck the superior slope about 15 ft. from the interior crest, and glanced upward into the shield from which it subsequently emerged. Other than the groove cut in the concrete by the projectile, the effect was slight and local. No fragments were to be seen on the terreplein, and the registering screen, placed in the rear, was apparently untouched.

The second shot struck the slope with a calculated velocity of 1 100 ft. per sec. about 10 ft. from the interior crest, glanced upward and embedded itself in the shield. The effect on the concrete was again slight and local; a groove formed about 2 ft. 7 in. from the interior crest with which it was connected by a crack about 4 ft. long; the registering screen was again unmarked.

The third shot struck, with the same velocity, the parapet 5 ft. from the interior crest, with the result that the concrete broke conically at a distance of about 2.25 ft. from the crest, the base lying in the plain of the inner face of the breast-height wall with its horizontal and vertical axes about 5 ft. long. The mass of concrete was not scattered but was carried bodily, and nearly horizontally, backward through the screen; no cracks other than the surface ones were apparent.

The fourth shot was with a full charge of 580 lb., and a striking velocity of about 2 000 ft. per sec. The gun was aimed so that the projectile struck the earth about 5 ft. outside of the concrete. A considerable cavity was found in the earth, and the shot entered the concrete apparently normal to and 3 ft. below the superior slope, penetrating to within 11 ft. of the interior crest, and was deflected to the right about 4 ft. 10 in., when it came to rest, with the base tilted upward, 15 in. below the slope. The concrete over the path of the shot, and for several feet on either side of the path, was broken into large blocks, each containing about ⅓ cu. yd. and less. No concrete was thrown into the emplacement by this shot, the interior face, apparently, was not deformed, but numerous cracks indicated that the whole mass of concrete was somewhat shattered.

The one shot against the second type of parapet was aimed so as to strike about 17 ft. outside of the interior crest with, approximately, the same angle of fall and a calculated striking velocity of 1 100 ft. A considerable quantity of earth was thrown out, and there was no evidence that the shot was deflected upward to any great extent during its passage through the earth. When the rails were struck, twenty-one were broken off about 8 ft. below the interior crest; the lower parts of the rails were bent back into the emplacement, remaining firmly embedded in the concrete at the bottom. The portions broken off were shattered into hundreds of fragments, some of which were hurled high in the air. The screen behind the parapet showed a hole, 15 ft. wide by 5 to 10 ft. high, and many scattered hits elsewhere. It was very apparent that no man could have stayed behind the parapet during the firing and lived.

As the result of these experiments, no change in the construction of the parapet has been made, other than in the reduction of 5 ft. in the thickness of the concrete.

Concrete replaced the cut-stone masonry of the old seacoast works, and while Rosendale concrete with 2 ft. of Portland finish was used in the earlier types of the present system, Portland concrete is now used exclusively. The largest dimensions of the crushed stone (the run of the crusher is preferred) is 1.5 in. Cutting into the concrete of some of the earlier batteries disclosed comparatively large open spaces about the boulders and blocks of stone which had been used with the concrete, which necessarily weakened the strength of the mass to resist penetration, and the practice of embedding boulders, etc., was discontinued. The arched ceilings of some of the earliest types gave way to horizontal coverings of I-beams and concrete, and this again was modified by the use of "steel-concrete" with various types of bars, or a combination of these with expanded metal. As the greatest danger to these ceilings is from the shock of striking projectiles, additional bars are often added to the upper part of the beam which forms the ceilings to the rooms. In the more recent types planes of weakness are provided, and yet even then, with the large masses used, the inequalities of setting and the resulting internal strains cause the development of cracks in the best and most carefully laid con-

crete, through which water percolates. Water-proofing of asphalt did not give satisfactory results, and this is replaced, in later constructions, by tarred paper or felt laid with tar pitch; the water-proofing is also carried under the foundation walls and up the side walls as far as may be necessary.

Water due to condensation, and to percolation through the cracks developed, as already noted, has been the cause of the greatest difficulty and annoyance in modern batteries. The cracking, in most cases, caused a sufficient disturbance of the water-proofing course to allow water to pass. Concrete in such large masses does not respond to rapid changes in temperature, and, as a result, in the humid atmosphere of the seacoast, the rooms of the emplacements were constantly wet during certain seasons of the year from condensation alone.

The difficulties, due to percolation, have been entirely eliminated by the use of better cement and better water-proofing material; breaks in the water-proofing course are obviated by laying it in a very poor mortar; leakage in earlier works is stopped by the application of elastic cements, of preparations of tar, etc., to the exterior surfaces.

In the old, stone forts, the magazines were not exposed to the exterior air as in the modern types; they were kept closed, and, as a consequence, condensation was practically unknown. In the modern types doors are left open regardless of the weather conditions, and water in the magazines follows. While care on the part of the garrison can to a great extent control the amount of condensation, such care is not always exercised, and, therefore, other means must be resorted to in order to keep the rooms dry. Heat is effective but expensive, and where heating plants have been installed, they have not been cared for properly as bursting pipes in winter attest. Experiments have been made, in the batteries already constructed, with linings of brick, tile, cork, magnesia boards, wood, etc., but with only partial success, the best results being obtained when an air space is left between the wall and the lining. Ventilation has been successful in some cases. In the more recently constructed batteries greater attention is paid to proper ventilation; the walls have been made thinner by the use of a large air space which practically separates the rooms from the adjacent masses of concrete, and the magazines are, prac-

tically, entirely detached rooms within the concrete. This permits the use of brick walls throughout, and if the interior face of the walls be of porous brick or tile which will absorb the moisture until the walls are heated to the corresponding temperature of the outside air, the room does not appear wet as in the old types. Experiments were made with various makes of brick and tiles to ascertain their relative porosity, but all were found to absorb more water than even saturated air contained, so that rapidity of absorption is a more important item than quantity. The air spaces, large enough to admit the passage of a man, are connected with the exterior air, and help to ventilate and heat the walls of the rooms. They are utilized for drainage, and through them are passed water pipes, electric ducts, etc.

Ammunition is handled as follows: powder from the magazines, by trucks and by hand; projectiles from the shell rooms, galleries, etc., by overhead trolleys and specially designed trucks. It is transferred to the level of the gun platform by cranes, platform lifts or chain hoists. Where cranes are used, the ammunition is transferred from trolleys or trucks to the cranes and delivered to trucks on the upper level. In the platform lifts, the truck with the ammunition is raised, and the truck taken directly to the gun. With the chain hoists, the ammunition is delivered by trolley to a specially designed table from which it is transferred to another table on the level of the loading platform by the hoist, and taken to the gun by the truck. The necessity for increased rapidity of fire, the difficulty of keeping the lifts in satisfactory working order, due largely to lack of proper attention, have led to the exclusive use of the chain hoist with electric-motor attachment, as well as hand-power, in all new work, and to its substitution for the other types, as funds become available, in older batteries.

Because of the great number of rapid-fire guns now carried by naval vessels, and the high rate of speed at which they can move, it is necessary for the defense to begin the artillery duel at as long ranges as the guns will permit, and in order that the fire from the batteries may be effective, the ranges and azimuths must be accurately determined. For this purpose some system of range finding becomes an essential adjunct. There are three systems: by the use of a vertical base, of a horizontal base, or of a combination of the two. The vertical base requires, for the instrument adopted

by the Board of Ordnance and Fortification at the outbreak of the Spanish War, a base of at least 60 ft., and while satisfactory results are obtained, on low sites, towers are necessary to secure the proper base, which make conspicuous targets for the enemy. This system requires water lining the ship, with proper corrections for tidal fluctuations. The horizontal base system avoids the use of towers even on low sites, is more accurate at long ranges if the stations are properly selected, but has the disadvantage of liability to mistakes in identification of the particular ship on which fire is to be directed. A combination of the two systems avoids this disadvantage, the vertical instrument is used to identify the target, and the ranges and azimuths are determined by the horizontal system. The writer has been informed that Warner and Swazey have perfected an instrument which gives accurate results up to 10 000 yd., working with a 25-ft. horizontal base; if this be the case, the horizontal base system is likely to be the one eventually adopted. Target practice is carried on annually at moving targets, with the range-finding systems already installed, and the results obtained are exceedingly gratifying.

Electricity is an important accessory to seacoast defense, no longer confined to the submarine-mine portion of the defense, as was the case some ten years ago. When first installed the plants were small and of capacity sufficient for lighting the rooms and galleries of a battery, or of one or more batteries where they happened to be grouped close enough for one small plant to do the work; this was then regarded as sufficient. In 1900, under orders of the War Department, individual battery plants were, thereafter, to be replaced by central power stations ample for all the purposes of defense, and when not in use for the fortifications, the plants were to light the buildings and grounds occupied by the garrison. After electricity was installed for lighting, its application was extended, until at present it is intended to supply motors for operating the ammunition hoists, for traversing and retracting the guns, and for running a small machine shop for minor repairs to guns, carriages, etc.; arrangements are also made for electric firing of guns and mortars, and electric current is supplied for operating search-lights and for the fire-control system.

Necessity for search-lights to guard mine fields against any attempt to destroy the mines has already been noted. But the use

of search-lights has been further extended to guard against surprise should night attacks be attempted by an enemy's fleet. It is questionable whether, with all the aids to navigation removed, as would be the case in time of war, an attempt would be made to enter a harbor after dark, and it is doubted by many; still, an adventuresome commander might not hesitate to do so, and it will be necessary for the defense to guard against the contingency. The adopted sizes of search-lights for defensive positions are 30, 36 and 60-in. diameters, depending upon the field to be illuminated. The 60-in. search-lights are to be used for observing the movements of a fleet in the offing, so that warning may be given of any attempt to approach; they are the searching lights. In case an advance is made the fire or battle commanders will assign certain ships to certain batteries, the commanders of which will illuminate the target by the smaller or fighting lights. Comparatively few lights have thus far been installed, due to lack of appropriations, but the matter has been carefully studied, and locations selected for the lights. Where they are in comparatively close proximity to central power stations the current will be supplied by such stations, but where the distance renders such transmission expensive, independent plants for each light will be installed.

The fire-control system consists of an elaborate arrangement of electrical communications for the transmission of orders, ranges, azimuths, etc. The defenses of any particular harbor are under the control of the battle commander. Various batteries are grouped together under a fire commander and each battery is in charge of the battery commander. The battle commander's station is electrically connected with the fire commanders' stations by telephone and telautograph. These stations are in turn connected with the battery commanders' stations and with the rapid-fire armament; with the former by telephone and telautograph, with the latter by telephone; and the battery commander is connected with the guns of his battery and with the horizontal base and the vertical base systems, when these are independent of his particular station, by telephones. The search-lights are connected with the battle commanders', fire commanders' and battery commanders' stations by telephones. Horizontal base stations are connected with each other by telephone. A written record is desirable, and while the telautograph gives this, it has not proven satisfactory during

recent maneuvers, and whether it can be made so as to be constantly reliable remains to be seen; comparatively few have thus far been installed. The necessary power for these communications is supplied by the central station.

As before stated, coast-defense vessels form an important part of a complete system of defense, but, as yet, no steps have been taken in the United States for procuring such vessels, if the monitors belonging to the navy are omitted from consideration. The same is equally applicable to torpedoes and submarine boats, although the Torpedo Board has recommended the purchase of a certain number of submarine boats for experimental purposes. The matter is in the hands of the Joint Army and Navy Board for consideration and the working out of the necessary details.

The question of land defense is a very important one, and the necessary plans have been prepared by the local engineer officers for giving proper protection to the defenses against any attempt which may be made by landing parties, and, in the event of war, it is expected that the necessary steps will be undertaken for their construction, using the field type of fortification. The matter is still under consideration.

Congress for the past ten years has been more liberal in its appropriations for coast defense, and, as a result, the heavy guns which are to be mounted under the approved projects are, to a great extent, already installed; and, in this respect, the coast is in a fair state of protection against attack; but these appropriations are gradually falling off, and, until another war scare, it is probable that comparatively little work will be done. At present the greatest difficulty is in the proper care of the guns already mounted. The Coast Artillery branch of the service, to which this belongs, is too small in numbers to man the guns now available, and the greatest danger now confronting the United States arises from this deficiency of properly trained men to fight the guns. At the outbreak of war a fleet could be off any important harbor within a few days, and long before it would be possible to organize the necessary volunteer force to properly man the guns, and even were this possible they would be ignorant of the technical knowledge now needed for the proper handling of the armament. This is a matter, however, which devolves upon Congress to rectify, and what steps will be taken to overcome this danger, remains to be seen.

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TRANSACTIONS.

INTERNATIONAL ENGINEERING CONGRESS.

1904.

LIVE LOADS FOR RAILROAD BRIDGES.

Congress Paper No. 3.

By HENRY W. HODGE, M. AM. SOC. C. E., New York City, U. S. A.

Discussion of the Subject by:

ALEXANDER ROSS, London, England.
GUSTAV LINDENTHAL, New York City, U. S. A.
ROBERT MOORE, St. Louis, Mo., U. S. A.
J. E. GREINER, Baltimore, Md., U. S. A.
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NOTE.—Figures and Tables in the text are numbered consecutively through the papers and discussion on each subject.

TRANSACTIONS

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INTERNATIONAL ENGINEERING CONGRESS.

1904.

Paper No. 3.

LIVE LOADS FOR RAILROAD BRIDGES.

By Henry W. Hodge, M. Am. Soc. C. E.

The weights of locomotives used by various railroads in the United States have been increasing so steadily, that even those most intimately connected with the design and use of bridges seldom realize what a complete change has taken place in these weights in the last few years. This increase, however, has been the cause of the constant replacement of iron and steel bridges by heavier structures. It will, perhaps, save a repetition of this needless expense, if those in charge of bridge matters look carefully into the increase that has been going on so constantly, and, from the data so obtained, form a fair estimate of how much farther and how rapidly this increase is to continue, so that they may have the bridges under their charge designed for loads such that the structures will not need to be condemned as "too light for traffic," within a period very much shorter than their natural lifetime.

A great deal has been written on the lifetime of steel bridges, but the theories advanced cannot be proved by practice, as, so far as the writer knows, all bridges replaced have been able to carry the loads for which they were designed. The writer believes, also, that bridges which are being built to-day for the loads now actually in use will seldom be allowed to stand for their natural lifetime, but

will be condemned because of insufficient strength to carry the loads of the future.

It is very hard to give any complete data covering the loads used by all railroads, but such loads as the writer has been able to gather from various sources are shown on Plates I and II. These diagrams show a very fair average of the loads generally used by the leading railroads of the United States, from 1886 to 1903.

It will be noted that most of these loads are locomotives of the "consolidation" type, as this is generally the heaviest loading, although many roads also show a locomotive of the "passenger" type, but as this last-named type has, usually, but two heavy axle loads, which are spaced much farther apart than for freight engines, such loads seldom come into use except in the design of floor systems. The writer, therefore, has not considered it worth while to tabulate loads of this class. These loads are not the actual weights of locomotives as used, but are the conventional loads adopted for designing bridges on the various railroad lines; and they were doubtless selected as fairly representing the heaviest locomotives in use at the time, or as providing a reasonable allowance for future increase. These loads have been shown in columns, by years, with the lightest loads at the top and the heaviest loads at the bottom of the columns.

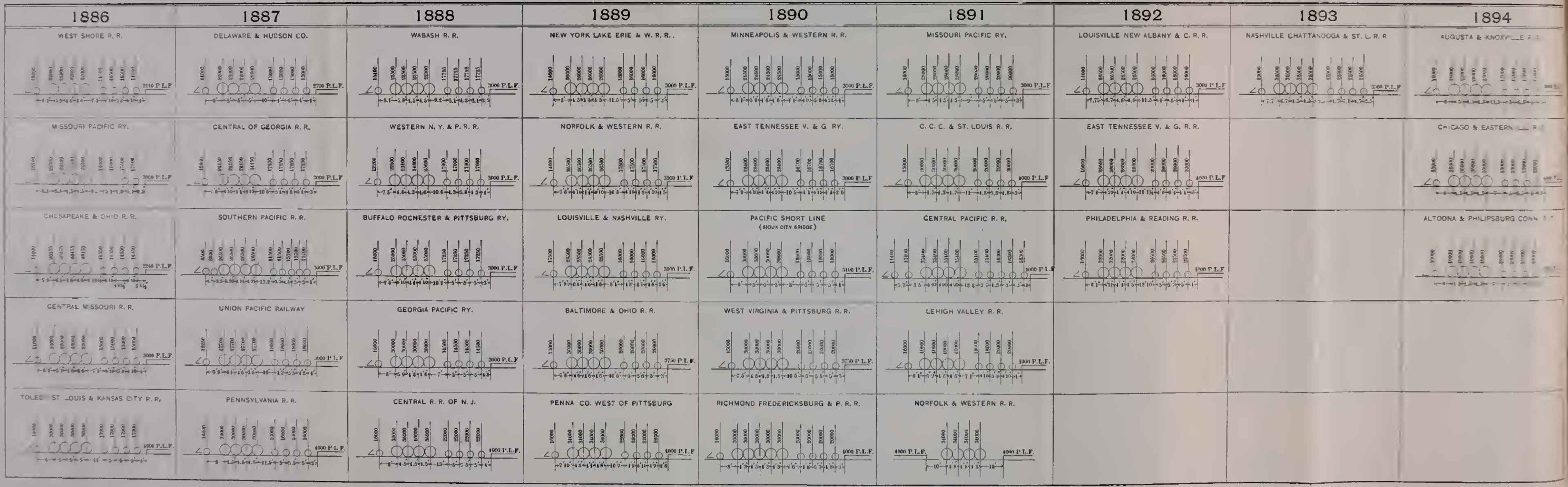
As already stated, it is not claimed that these loads show absolutely the lightest and heaviest loads in use at the time, but they are the lightest and heaviest loads which the writer was able to find from an examination of about two hundred different loads, though, of course, narrow-gauge roads or roads built for specially light traffic are not included. A general inspection of Plates I and II will show how steady has been the increase in loads, and, taking the maximum weight on one driving axle as an index of the whole load, the rate of increase is found to be as follows:

Year.	Maximum weight on driving axle.
1886	30 000 lb.
1888	36 000 "
1891	40 000 "
1896	48 000 "
1899	50 000 "
1900	55 000 "
1903	66 000 "

PLATE I, VOL. LIV, PART A.
TRANS. AM. SOC. CIV. ENGRS.
INTER. ENG. CONG., 1904.
HODGE ON
LIVE LOADS FOR RAILROAD BRIDGES.

LIVE LOADS FOR RAILROAD BRIDGES FROM 1886 TO 1894.

LOADS CONSIST OF TWO ENGINES LIKE THE ONE SHOWN.



It will thus be seen that the maximum axle loads have more than doubled in the last 17 years, and while this may not be true of every railroad, the statement is probably not far from correct.

Taking the increase on individual roads, as exhibited in Plates I and II, for the four roads which show the largest number of different loadings, the results shown in Table 1 are found:

TABLE 1.

Wabash R. R.		Missouri Pacific R. R.		Norfolk & Western R. R.		Southern Pacific R. R.	
Year.	Maximum axle load, in pounds.	Year.	Maximum axle load, in pounds.	Year.	Maximum axle load, in pounds.	Year.	Maximum axle load, in pounds.
1888....	22 500	1886...	22 500	1889...	26 500	1887...	26 500
1895....	35 000	1891...	28 000	1891...	34 000	1896...	48 000
1898....	40 000	1895...	32 000	1897...	40 000	1901...	50 000
1901....	50 000	1901...	40 000	1898...	45 000	1902...	55 000
		1902...	50 000	1903...	56 000		
Increase, 122% in 13 years.		Increase, 122% in 16 years.		Increase, 111% in 14 years.		Increase, 108% in 15 years.	

It is to be noted that some of the most prominent railroads made considerable increases in axle loads at about the earliest dates shown on the diagrams on Plate I, so that their rate of increase has not been so rapid during the last 15 years.

Having thus obtained a fair average of the course of this increase during the last 17 years, a rational estimate may be formed of what may result in the future. In a previous paper on this subject,* the writer has given his ideas on this subject, which he will repeat here, with some enlargement, gained from the discussion on that paper, and from the inspection of loads now in use by the larger railroad systems.

That economy in operating expenses has been greatly increased by hauling longer and heavier trains by heavier locomotives, is a

* *Transactions*, Am. Soc. C. E., Vol. LI, p. 105.

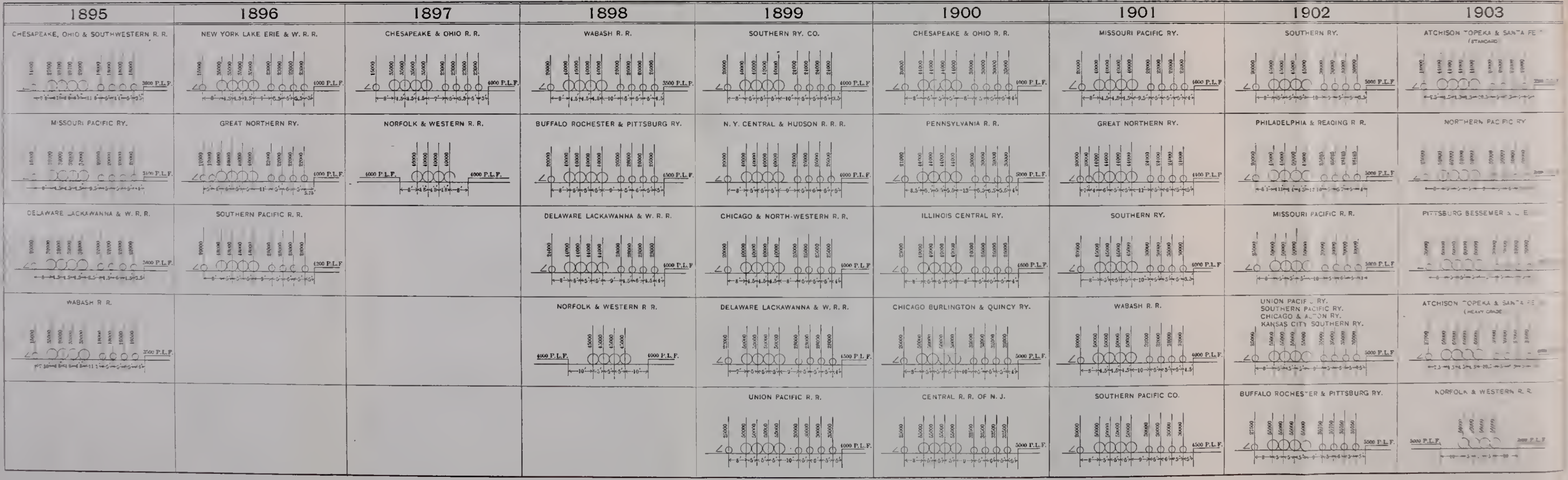
fact beyond dispute, and there is no evidence that this economy cannot be still further increased by continuing this course of moving larger units. Unless the maximum economy in this direction has been reached, it may be assumed, as a certainty, that the operating departments of railroads will continue to increase their live loads. even if it does necessitate the continued replacement of bridges.

It has been stated that trains have now reached such a length that it will be impracticable to haul longer ones; but, if this statement be accepted as a fact, there is no reason why the carrying capacity of each car cannot be largely increased without altering its length. When it is remembered that the modern steel coal car carries 50 tons, when only a few years ago coal cars of the same length carried only a maximum of 30 tons, and when the possibilities of building cars to carry 100 tons of iron ore in an over-all length of 33 ft. are seen, it seems evident that the weight of trains can be very largely increased without increasing their length. Of course, this ore car, last mentioned, is an extreme case, and it naturally cannot be taken as applying to roads which do not carry this class of traffic, but it proves that very much larger loads, than those for which most cars are now built, can be carried, if this be found economical. Even if the statement that the present length and width of cars has reached a maximum is accepted, there still remains ample room to increase the height of cars sufficiently to increase greatly their capacity. It is in this direction that the increase is being made, as all have noticed, in the greatly increased height of the sides of modern open cars. Even in box cars there is ample room for an increase in height, as vertical clearances on all roads have been designed to allow the brakeman to stand on top of the cars, and (now that the use of air-brakes has become so universal on freight trains) the day of the necessity for clearance for men on top of cars has practically passed. Assuming, then, that there is, at least, a possibility of increasing the weight of trains, the next objection is, that the weight of locomotives cannot be increased because the roadbed will not carry heavier loads than are now in use, but this objection has been raised constantly for the last twenty years, while we have seen the engine loads doubled, and the roadbeds still carrying them satisfactorily. Of course, the weights of rails have been greatly increased, and the ballasting has been made heavier, but there is no visible reason why a continuation of

PLATE II, VOL. LIV, PART A.
TRANS. AM. SOC. CIV. ENGRS.
INTER. ENG. CONG., 1904.
HODGE ON
LIVE LOADS FOR RAILROAD BRIDGES.

LIVE LOADS FOR RAILROAD BRIDGES FROM 1895 TO 1903.

LOADS CONSIST OF TWO ENGINES LIKE THE ONE SHOWN.



improvement in this direction should not go on. The writer cannot see any sufficient reason why the permanent way cannot be increased in carrying capacity as fast as is found desirable.

If, therefore, there is a desirability and a possibility of increasing the weight of trains, and if the roadbed can be designed so as to carry any locomotive desired, the final maximum attainable must lie in the possibility of increase in the weight of the locomotive itself, and, in the writer's opinion, this is the point as to which the maximum has been most nearly reached.

The heaviest locomotive now in actual use, so far as the writer has been able to learn, is the "Decapod" type now in use on the Atchison, Topeka and Santa Fé Railroad, which has its weights distributed as in Fig. 1, which shows a weight of 267 000 lb. for the

FIG. 1.

locomotive and 134 000 lb. for the tender, or a total of 401 000 lb., complete, on a total wheel base of 59 ft. 6 in.

The Pittsburg, Bessemer and Lake Erie Railroad is using consolidation locomotives with still heavier axle loads distributed as in Fig. 2, which shows a weight of 250 300 lb. for the locomotive and

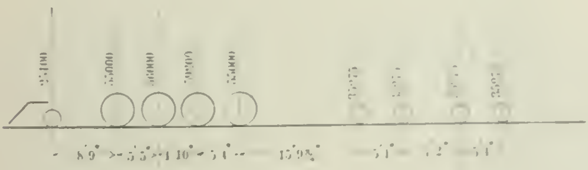


FIG. 2.

141 100 lb. for the tender, or a total of 391 400 lb. on a total wheel base of 57 ft. 11¾ in.

The height of these large locomotives has nearly reached a maximum for present railway clearances, as will be seen from the view of a Decapod locomotive of the Atchison, Topeka and Santa Fé Railroad in Plate III, where the top of the boiler comes almost up to the clearance line.

The width, also, is about as great as present clearance will permit, and the driving wheels cannot probably be much reduced in diameter, so as to give more space above the axles for additional weight, so that there does not seem to be much probability of greatly increasing the weight of these locomotives, unless the present railroad clearances are increased, which is impracticable.

However, the writer has asked the opinions of expert mechanical engineers, engaged in the design of locomotives, what, in their opinion, was the possibility of further increase in axle loads within the present clearance dimensions, and they seem to think it probable that the present maximum loads can be increased 15 per cent. An addition of this amount would give a maximum locomotive load about as shown in Fig. 3.

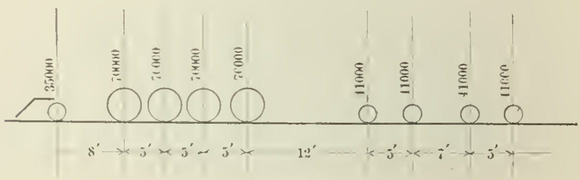
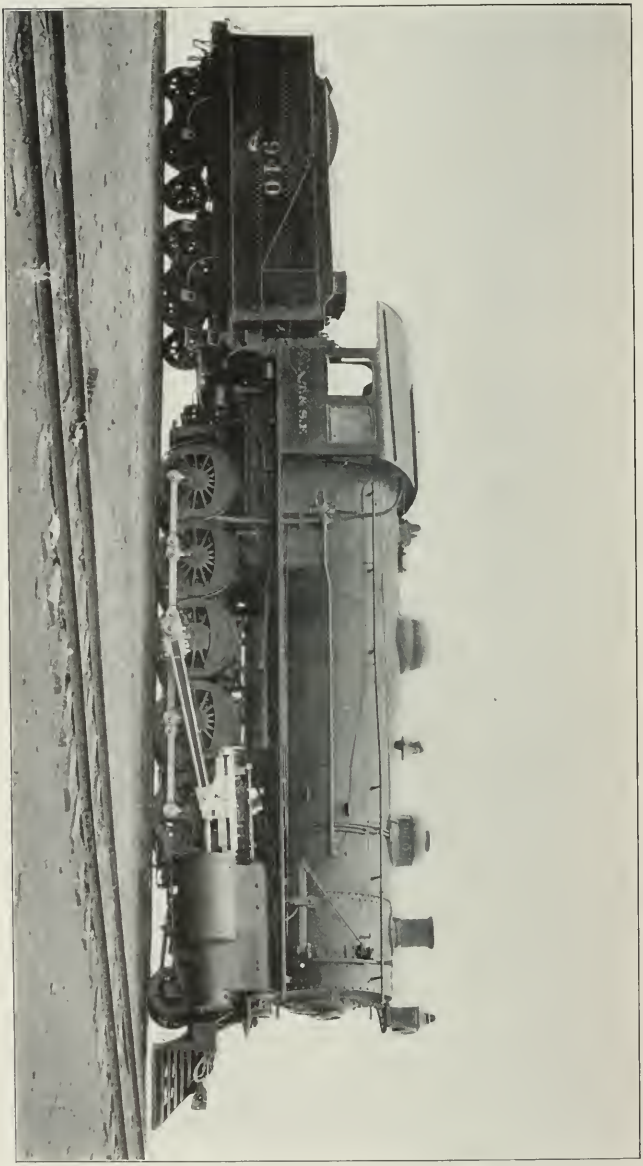


FIG. 3.

This theoretical loading does not represent a great increase over the loads now in use, but, in the opinion of the writer, it will cover the probable maximum locomotive load for a long period to come; and while it will, of course, be entirely too heavy for the majority of railroads, where no such heavy traffic will ever be required, it will

PLATE III, VOL. LIV. PART A.
TRANS. AM. SOC. CIV. ENGRS.
INTER. ENG. CONG., 1904.
HODGE ON
LIVE LOADS FOR RAILROAD BRIDGES.



DECAPOD LOCOMOTIVE, A., T. & S. F. R. R. 1903.

probably be reached by those roads having heavy grades and hauling the heaviest classes of traffic.

It only remains, now, to look into the possibilities of train loads to follow the maximum locomotive loads. This question does not appear to be as uncertain as that of locomotive loads, since it is known that the modern steel coal car carries 100 000 lb. and weighs about 40 000 lb. with an over-all length of about 35 ft., making an average load of 4 000 lb. per lin. ft.

But the roads which carry iron ore have cars much heavier than this. The heaviest cars in use on the Monongahela Connecting Railroad, of Pittsburg, Pa., carry 200 000 lb., and weigh 40 000 lb., the axle loads being spaced as shown in Fig. 4.

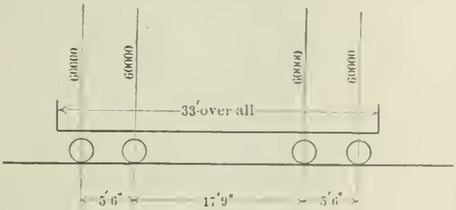


Fig. 4.

This makes an average load of about 7 300 lb. per lin. ft., and if allowance be made for the same increase in this uniform load of 15%, a maximum of 8 400 lb. per lin ft. would be reached.

As before stated, it would be ridiculous to specify any such loads as those arrived at above for most railroads, but it is not the writer's intention to advocate any such loads for general use, nor the use of a uniform set of loads for all roads, as it is evident that each road must adopt such loads as its traffic warrants. The loads above suggested are only given as the probable maximum for roads carrying the most extreme traffic.

However, the writer would strongly advise that all new bridges on any road be designed for loads somewhat heavier than those in actual use at the time the structure is designed, as this increase in rolling loads has been going on steadily, and, from the foregoing facts, he is unable to see that the process has stopped. A small extra expense now, may thus save the throwing away of the structure and its entire reconstruction on a heavier basis within a comparatively short time.

It would appear advisable, therefore, for roads expecting to carry the heaviest class of traffic to design their bridges for a moving load about as indicated in Fig. 3. Any road which expects to do an ordinary traffic, or carry the freight delivered to it by other large

systems, will, in the writer's opinion, be unwise to use a load of less than 50 000 lb. on drivers, followed by 5 000 lb. per ft., as such loads will probably be quite universal in the very near future, and, even now, are exceeded on several of the large railroad systems.

TRANSACTIONS

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INTERNATIONAL ENGINEERING CONGRESS,

1904.

DISCUSSION ON

LIVE LOADS FOR RAILROAD BRIDGES.

By Messrs. Alexander Ross, Gustav Lindenthal, Robert Moore, J. E. Greiner, C. D. Purdon, W. M. Camp, Charles S. Churchill, Albert Reichmann, J. M. Johnson, A. F. Robinson and Henry W. Hodge.

Alexander Ross, M. Inst. C. E., London, England.* (By letter.) Mr. Ross. —The modern traffic requirements of Great Britain are similar to those stated by Mr. Hodge to be operating in America, and continual efforts are being made to increase the train load, necessitating more powerful and heavier engines.

Engineers in Great Britain, who have charge of the maintenance of bridges and permanent way, have for years been seriously considering the effects caused by the increased weights of locomotives upon the structures under their charge, and opinion is gradually tending toward the view that a hypothetical or type engine should be agreed upon, the weights of which would not be likely to be exceeded by any locomotive which could be reasonably devised, the weights given by such an engine to be adopted by engineers in designing the bridges of the future.

In Great Britain the load gauge is so defined that a limit in the weight of engines can with some certainty be foreseen, whereas in America the gauge is so ample that some difficulty may be met with on this account.

Some of the examples given by Mr. Hodge, and the typical engine

* Chief Engineer, Great Northern Railway of England.

Mr. Ross. suggested by him, exceed in weight any engine in use in this country or likely to be adopted.

At the Engineering Conference held in London in June, 1903, the writer prepared a note on this subject, which was read and discussed, in which he gave a table of equivalent distributed live loads derived from maximum bending moments for a single line of way given by selected engines then running, and he now attaches a copy of that table, to which have been added the equivalent loads given by such a typical engine as that suggested at the London meeting, in the belief that acting on such weights would meet the requirements of this country as long as the existing load gauge is unaltered.

To the table the equivalent loads for the Decapod engine described as the heaviest on American roads, and for the type engine proposed by Mr. Hodge as a maximum, have also been added. The table shows clearly the enormous weights being carried by and proposed for American lines as compared with British railways as their maximum.

It is the usual practice in Great Britain to couple up engines for various purposes, such as running between the engine yards and stations in large towns, and as this causes the greatest stresses on the bridges run over, it is customary with us to take a train of engines as the measure of the weights to be provided for, and the accompanying table has been prepared on that basis.

The effects of the various loads have been plotted, and an equalising curve has been drawn from which the figures given have been obtained by scale. All the figures have had 2½% added for contingencies, and the weights are given in English tons of 2 240 lb.

Mr. Hodge toward the end of his paper refers to train loads, and expresses the opinion that these are likely to be further increased on some lines up to 8 400 lb. per lin. ft., and that weights of 138 000 lb., or 61½ tons, may be concentrated on bogies at each end of freight wagons.

These would then become almost as heavy as engines and form an additional reason why the latter should be taken as the gauge of the weights to be provided for.

In Great Britain the wagons with heaviest loads are of 40 tons capacity and weighing 16 tons each on a wheel base of 40 ft. 6 in.

The writer has noted the great difference in the weights of engines used on the different lines in America and Mr. Hodge's suggestion that some lines should provide for 70 000 lb. on drivers and some for 50 000 lb., and doubts the wisdom of underestimating the provision for the latter.

Engines are short-lived and are constantly being replaced and their number increased, whereas a bridge should be constructed to

Mr Ross

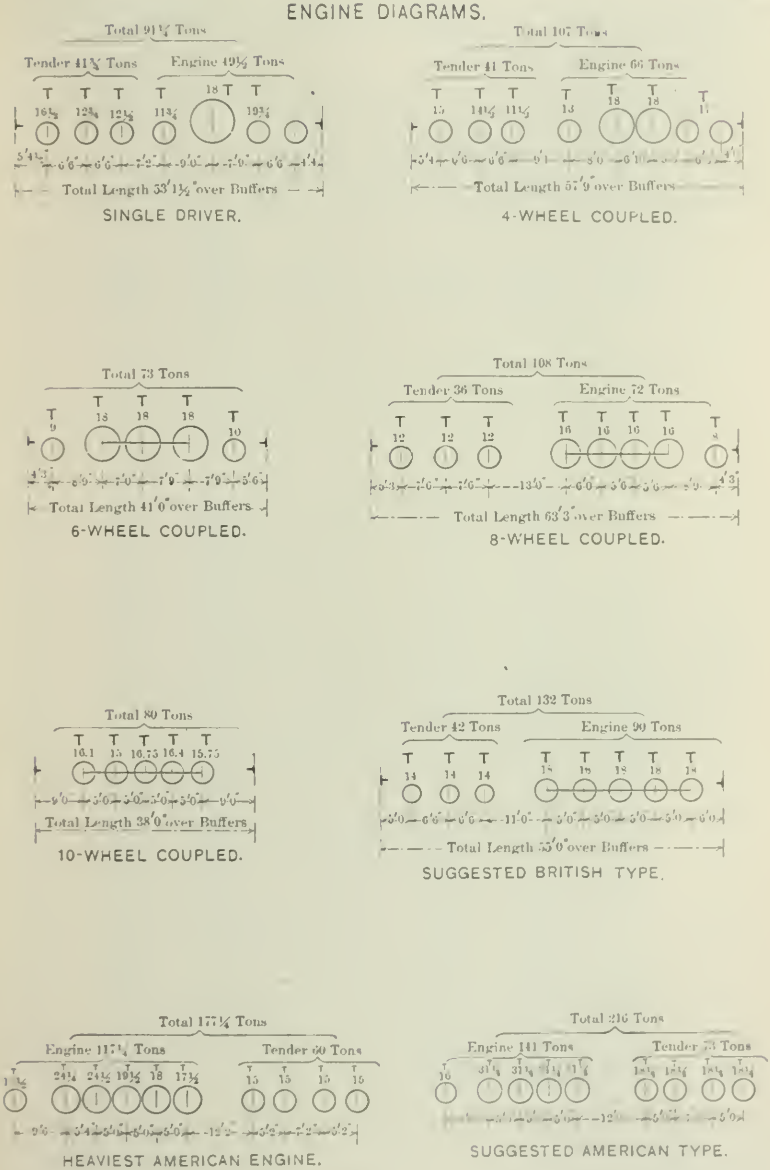

ENGINE DIAGRAMS.
Total 91¼ Tons
Tender 41¾ Tons
Engine 49½ Tons
Total Length 53′ 1½″ over Buffers
SINGLE DRIVER.
Total 107 Tons
Tender 41 Tons
Engine 66 Tons
Total Length 57′ 9″ over Buffers
4-WHEEL COUPLED.
Total 73 Tons
Total Length 41′ 0″ over Buffers
6-WHEEL COUPLED.
Total 108 Tons
Tender 36 Tons
Engine 72 Tons
Total Length 63′ 3″ over Buffers
8-WHEEL COUPLED.
Total 80 Tons
Total Length 38′ 0″ over Buffers
10-WHEEL COUPLED.
Total 132 Tons
Tender 42 Tons
Engine 90 Tons
Total Length 55′ 0″ over Buffers
SUGGESTED BRITISH TYPE.
Total 177¼ Tons
Engine 117¼ Tons
Tender 60 Tons
HEAVIEST AMERICAN ENGINE.
Total 216 Tons
Engine 141 Tons
SUGGESTED AMERICAN TYPE.

FIG. 5.

Mr. Ross. last for a long time and a liberal provision made for the future, rather than risk the expense of renewal, it being borne in mind that the most economical time to give the full strength to a bridge is when it is being constructed.

Equivalent Distributed Live Loads Derived from Maximum Bending Moments for a Single Line of Way.

Referred to by Mr. Ross, Engineering Conference, London, June, 1903.

Span, in feet.	Single driver.	4-wheel coupled.	6-wheel coupled.	8-wheel coupled.	10-wheel coupled.	Type engine suggested by Mr. Ross.	American Decapod, A., T. & S. F. Ry.	Type engine suggested by Mr. Hodge.
	Tons distributed.	Tons distributed.	Tons distributed.	Tons distributed.	Tons distributed.	Tons distributed.	Tons distributed.	Tons distributed.
10......	36.9	36.9	36.9	34.6	39.9	41.5	54.1	72.2
15......	38.1	46.6	48.8	50.3	55.8	61.5	76.2	105.3
20......	44.0	57.6	56.2	63.5	68.9	79.0	94.0	130.8
25......	51.5	65.4	66.3	73.8	83.7	96.0	110.2	153.0
30......	61.2	73.6	74.7	83.2	98.5	110.70	125.4	173.1
35......	71.1	82.6	84.0	91.4	105.9	121.45	140.0	190.05
40......	80.4	89.0	92.4	98.8	115.3	130.0	154.0	205.6
45......	90.0	95.6	99.0	105.6	120.2	139.05	166.95	217.8
50......	99.0	105.3	104.0	112.3	125.0	148.0	178.5	230.0
55......	107.8	115.0	111.1	119.0	130.7	157.85	189.75	241.45
60......	116.0	124.8	117.6	126.0	136.3	168.6	198.6	254.40
65......	125.5	133.9	124.8	133.0	147.6	178.75	206.7	267.8
70......	135.3	142.8	132.3	140.5	158.8	189.0	215.6	282.2
75......	144.0	151.5	141.8	150.0	169.5	198.75	222.75	296.25
80......	152.7	160.4	150.4	159.2	180.8	208.0	231.20	312.0
85......	162.0	168.3	159.8	168.3	192.1	217.6	239.7	327.25
90......	172.0	176.4	168.3	176.4	202.5	228.15	248.4	342.0
95......	180.4	184.3	177.7	184.3	213.8	239.40	258.4	356.25
100......	188.6	193.3	186.0	192.0	223.7	251.7	270.0	370.0
105......	197.4	201.6	195.3	201.6	235.2	261.28	281.4	383.25
110......	206.8	211.2	204.6	211.2	246.4	276.86	292.6	396.0
115......	216.2	220.8	213.9	220.8	257.6	289.45	304.75	409.4
120......	224.4	230.4	223.2	230.4	267.6	302.05	317.4	424.8
125......	233.8	240.0	232.5	240.0	278.7	314.62	330.0	442.5
130......	243.1	249.6	241.8	249.6	289.9	327.20	343.2	460.2
135......	251.1	259.2	251.1	259.2	301.1	339.79	355.05	477.9
140......	260.4	268.8	260.4	267.4	310.8	352.38	368.2	495.6
145......	268.3	278.4	269.7	277.0	321.9	364.67	381.35	513.3
150......	277.5	288.0	279.0	286.5	330.0	376.8	393.0	531.0
155......	285.2	297.6	288.3	296.1	342.6	389.05	406.1	548.7
160......	294.4	307.2	297.6	305.6	353.6	400.8	419.2	566.4
165......	302.0	316.8	306.9	315.2	363.0	412.83	432.3	584.1
170......	309.4	326.0	316.2	323.0	374.0	425.17	443.7	601.8
175......	316.8	336.0	325.5	332.5	385.0	437.5	456.75	619.5
180......	324.0	345.6	334.8	342.0	396.0	450.0	468.0	637.2
185......	331.2	355.2	344.1	351.5	405.0	460.65	481.0	654.19
190......	338.2	364.8	353.4	361.0	416.0	473.10	494.0	672.6
195......	345.2	374.4	362.7	370.5	427.1	483.6	507.0	690.3
200......	352.3	383.2	370.0	378.0	435.3	494.0	518.0	708.0

Mr. Lindenthal. Gustav Lindenthal, M. Am. Soc. C. E., New York City. (By letter.)—In discussing live loads in connection with the subject of wheel concentration, at the Convention of the American Society of

Civil Engineers, 1899,* the writer pointed out that the limit to the increase of wheel loads would be largely determined by the hardness and strength of the steel in rails and wheel tires, and the writer's opinion then expressed was that that limit would be reached with a load of 30 000 lb. per wheel, corresponding to a pressure of 60 000 to 75 000 lb. per sq. in. on the wheel tire and rail, which is close to the existing elastic limit of hard steel. Mr. Lindenthal.

It appears that already there are locomotives in use with pressures on the driving wheel of 33 000 lb., and Mr. Hodge contends that 70 000 lb. per axle, or 35 000 lb. per wheel, are within sight.

While thus it is true that in the course of four years a prediction as to wheel loads, which then appeared to be extreme, is already surpassed, the writer is nevertheless convinced and agrees with Mr. Hodge that the maximum has been reached, or will be shortly, and for the reason stated four years ago. The limit is set not so much by the strength of the steel rail as by that of the wheel tire. Heavier wheel loads will cease to pay when the cost of wheels and their maintenance becomes uneconomical.

Extremely heavy locomotives are not the rule on all railroads, but as one of them may occasionally pass over almost any railroad, the floor system particularly should be proportioned to carry them safely. It is not necessary for that purpose that the unit stresses should be kept uniformly low. It will suffice to adjust the unit stresses in proportion to the probability of such extreme load. Where this unit stress, for instance, is usually taken at 15 000 lb., with reductions for impact, it may suffice for most railroads to take that unit stress at 20 000 lb. for the same extreme load. In that way the designer will have better judgment as to the floor design and connections than he would by assuming a lower wheel load at the same unit stress of 15 000 lb.

It has been found that the floor system which has been properly proportioned for the vertical loads of locomotives has suffered from the horizontal forces that are transmitted into it from the traction of the locomotive.

With the probable maximum locomotive given by Mr. Hodge, that tractive force transmitted through the rail and ties into the stringers and floor beams, and then into the truss, would be 70 000 lb. in a length of 15 ft. That means that the floor beam may be subjected to a load transverse to its axis so great as to rupture the flange plates and angles crosswise. This has actually happened on bridges that had been designed for heavy freight traffic.

This points to the necessity of designing the floor system for a larger horizontal force than that usually provided in specifications as resulting from the braking of a freight train. This large trac-

* *Transactions*, Am. Soc. C. E., Vol. XLII, p. 189.

Mr. Lindenthal. tion force of the locomotive is a concentrated load acting horizontally in the same sense as the weight upon the drivers is a concentrated load acting vertically. Floor beams amply strong enough for the vertical loads are frequently weak transversely for the horizontal loads. The writer mentions this as an additional argument for ample strength in the floor system, which suffers more than any other part of the bridge from the increasing axle loads.

Mr. Moore. ROBERT MOORE, PAST-PRESIDENT, AM. SOC. C. E., St. Louis, Mo.—Mr. Lindenthal's suggestion of allowing a somewhat high unit stress for unusual loads is an important one. Twenty thousand pounds per square inch for medium steel, for example, is entirely safe for occasional loads, though one might not like to use it for loads that are often repeated. This is especially important in the examination of bridges on branch or light traffic lines, which may be called upon now and then to take heavy special loads.

Mr. Greiner. J. E. GREINER, M. AM. SOC. C. E., Baltimore, Md. (By letter.)—In connection with this subject, the following conditions are conceded:

a.—Weights of locomotives and cars have been increasing steadily.

b.—Limit has not yet been reached, but is in sight.

c.—Bridges are replaced mainly on account of increase in power and not because they fail under loads for which they were designed, except when their maintenance has been neglected, or their construction has been defective.

d.—Bridges designed in accordance with the best practice at the present time are capable of carrying with perfect safety, live loads considerably in excess of the loads for which they were designed.

e.—It will never be necessary to run excessively heavy power over a great many low-grade railroads, and excessively heavy power is usually confined within certain districts or territories.

f.—Engines of the heaviest class have a much longer wheel base than in the consolidation type, which is usually specified, and the resulting strains are not in direct proportion to the weight when one class of engine is compared with another.

Regarding Condition *a*, Fig. 6 illustrates this condition as it has existed on the Baltimore and Ohio Railroad since 1835. The weights of locomotives given are the actual weights of engines which were or are in service, with the exception of the year 1905, which year includes the Mallet articulated type, weight 323 000 lb., exclusive of tender, one of which was recently purchased. The B. & O. expects to put this engine in actual operation during 1904, or early in 1905.

Regarding Condition *b*, the heaviest engine in actual service, as given by Mr. Hodge, weighs 267 000 lb., without tender. This is a

Mr. Greene.

DIAGRAM.
SHOWING INCREASE IN WEIGHTS OF LOCOMOTIVES
SINCE 1835.

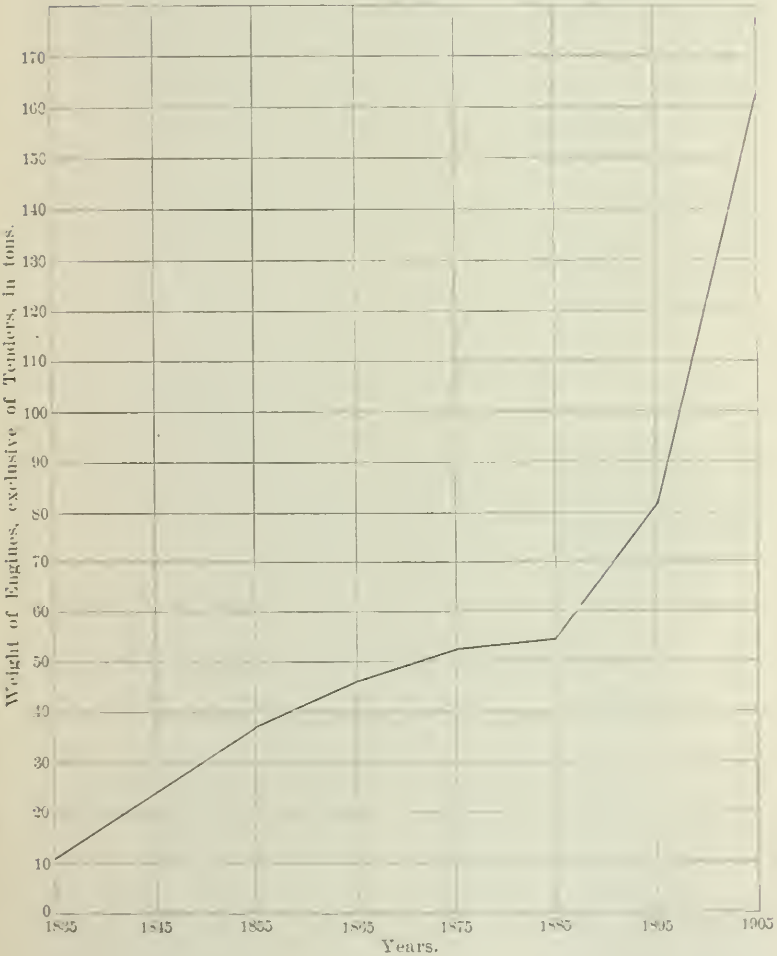


Fig. 6.

Mr. Greiner. Decapod. The heaviest consolidation weighs 250 300 lb. As already stated by the writer, the Baltimore and Ohio Railroad has a Mallet articulated type of engine, which weighs 323 000 lb. Here is already a considerable increase in weight since the writing of the author's paper. These weights may be still further increased, if found necessary; it is a question of economical operation, but experience with this heavy power shows that the results obtained are not proportional to the increased cost of operation and maintenance. Unless engines can be designed so that a saving in operation will result from their use, it stands to reason that these heavy types are isolated, and, with a few more experiments with some little increase in weight, will eventually be abandoned. It is the writer's opinion that the practical limit in weight of the consolidation type is in the neighborhood of 250 000 lb., of the Decapod about 280 000 lb., and of the articulated type, 350 000 lb., and this limit will be due to the fact that the increased cost of maintenance of equipment will not be offset by the results obtained.

Regarding Condition *c*, this was fully discussed in the writer's paper on "The Life of Railroad Bridges."*

Regarding Condition *d*, this was also fully discussed in the paper on "The Life of Railroad Bridges." A bridge designed in accordance with the best present practice, with full allowance for impacts and vibrations, with proper provision in counters for excess loads and with lateral system designed to resist lateral motion, will stand up indefinitely under a loading 50% in excess of that for which the bridge was designed, without undue deflection or motion, provided, of course, the structure is properly maintained.

Regarding Condition *e*, there is an economical limit to the length of trains. This limit has practically been reached on low-grade lines, and, on easy grades, the maximum number of cars can be handled without resorting to the use of excessively heavy engines. These heavy engines are economical, therefore, only for hauling this maximum length of train over heavy-grade districts, where it would be necessary to double up the lighter engines unless the heavier engines were resorted to. It is well recognized that the heavier the power the more the maintenance of way and equipment is going to cost, and it is therefore not good economy to run power much in excess of that necessary to haul the maximum train loads.

It would appear from the above, therefore, that the heaviest engines in the future will be located only on heavy-grade divisions. The proportion of them will be very small in comparison with the lighter class of engines which are in use on easy-grade divisions.

Regarding Condition *f*, in comparing the strains in bridges produced by the different types of engine mentioned by the author with

* *Transactions*, Am. Soc. C. E., Vol. XXXIV, p. 294.

Cooper's E-50 Class, it will be noted that the Decapod engine in Mr. Greiner. use on the Atchison, Topeka and Santa Fé Railroad, which weighs 267 000 lb., exclusive of tender, will not strain bridges more than 2 or 3% in excess of Cooper's E-50 for spans under 60 ft., and for greater spans the excess will be less.

The Pittsburg, Bessemer and Lake Erie Railroad consolidation engines, weighing 250 000 lb., exclusive of tender, give strains but slightly in excess of Cooper's E-50 for spans under 60 ft., and practically no excess for span lengths over that.

The consolidation type suggested by the author, shown in Fig. 3, which weighs 315 000 lb., gives strains about 40% in excess of Cooper's E-50 for spans under 30 ft., and a less per cent. for greater spans.

The Mallet articulated type of engine, recently purchased by the Baltimore and Ohio, which weighs 323 000 lb., gives strains but slightly in excess of Cooper's E-50 for spans below 22½ ft., gives strains less than Cooper's E-50 for spans between 22½ and 40 ft., and strains but slightly in excess for spans over 40 ft.

In view of what has been said above, the question arises as to what weight of engine and what type should be used in specifications, and it would appear that, inasmuch as there are quite a number of different types in use at the present time and that these types may be still further modified in the future, one type should be settled upon as a basis for the calculation of bridges. In the writer's opinion, the consolidation type should be adhered to and its weight should be taken at such an amount that if a structure should be designed in accordance therewith, it will be amply strong to take care of heavier engines of a different class which have their weights distributed over a longer wheel base.

The writer has been using in the Baltimore and Ohio specifications for the past two or three years Cooper's E-50 Class of engine, followed by 5 000 lb. per running foot, uniformly distributed. This engine weighs 225 000 lb., exclusive of tender. Bridges designed for this engine in accordance with the present practice would ride well and stand up well under a similar class of engine weighing 337 500 lb., followed by a train load of 7 500 lb. per running foot. As consolidation engines of this weight and of the same wheel base as Cooper's E-50 Class cannot be constructed and are impracticable, it stands to reason that bridges designed for Cooper's E-50 Class will never be subjected to anything like 50% excessive strains, as the heavier engines will have their weight distributed over a much longer wheel base, and, in all probability, no engine will ever be designed which would give strains over 25% in excess of those caused by Cooper's E-50, and inasmuch as these heavy engines, as stated above, would only be built for special service in particular

Mr. Greiner. places, it appears to the writer that Cooper's E-50 Class of engine, followed by 5 000 lb. per running foot, is as heavy loading as should be adopted for general bridge designing.

Mr. Purdon. C. D. PURDON, M. AM. SOC. C. E., St. Louis, Mo.—There are one or two interesting points in these discussions.

Mr. Hodge's paper gives the weight of the heaviest locomotive now in actual use, the Santa Fé Decapod, which has axle loads of 54 200, 54 800, 44 000, 40 000 and 44 000 lb. on spacing of 5 ft. 4 in., 5, 5 and 5 ft., respectively, and Mr. Greiner's discussion shows that on certain spans this is about the same as Cooper's E-50 Class.

Mr. Lindenthal considers that a weight of from 30 000 to 33 000 lb. on a wheel is about the limit of the tires and rails.

The speaker does not suppose that the engine shown in the Santa Fé specifications with a weight of 66 000 lb. per axle could be built on the spacing of 4½ ft. shown—but even then this engine would increase the strains on a bridge from 50% on a 20-ft. span down to 13% on a 200-ft. span, and as the Santa Fé unit strain is 9 000 lb. on bottom flange of plate girders, even this increase would be safe.

Taking all these discussions together, they seem to indicate that a loading of Cooper's E-50 is about as high as is necessary to cover all present engines.

Mr. Camp. W. M. CAMP, M. AM. SOC. C. E., Chicago, Ill. (By letter.)—In his remarks on the probable increase in the weight capacity of freight cars, Mr. Hodge states that "there still remains ample room to increase the height of cars sufficiently to increase greatly their capacity." On the assumption that "the day of the necessity for clearance for men on top of cars has practically passed," he thinks that there is ample room for increase in height, even in box cars, owing to the fact that vertical clearance on all roads has been designed to allow brakemen to stand on top of cars.

While the writer is not inclined to dispute the proposition that the carrying capacity of freight cars may yet be increased considerably, he is doubtful whether this should be done at the expense of vertical clearance. There are reasons for the presence of trainmen on top of freight cars other than for the purpose of setting brakes, which was a necessity before the general use of air-brakes on freight cars, as Mr. Hodge remarks; and as these reasons are not likely to disappear, a sufficient clearance over the cars for trainmen to walk the tops of the same while trains are in motion is a desirable provision for railway operation. Otherwise, it would be necessary to erect telltales over the track near all overhead structures to warn the men of danger. The construction of such devices is a matter of considerable expense, and their maintenance requires a great deal of vigilance to keep them in effective working

order. Sufficient clearance over the cars for trainmen to stand erect on the same is preferable, in any case, from an operating standpoint. Owing to the interchange of traffic, a change in such a condition of operation by one railway system, or a few systems, at most, would make it obligatory for practically all the railway lines of the country to erect warning signals at such of their overhead structures as would become dangerous to trainmen from the increase in height of cars, as proposed. Mr. Camp.

There is another reason why the height of cars should not be materially increased above the present standard dimensions in practice, and that is the possible raising of the center of gravity of the load carried by extending the limit of the vertical capacity of such cars. On some roads the height of cars has already been carried too far, for this reason, and this particularly applies to increasing the height of "the sides of modern open cars," to which practice Mr. Hodge especially calls attention, and evidently with approval. The highest of the modern hopper-bottom cars of steel construction, for carrying coal and ore, are already subject to excessive lateral sway, as maintenance-of-way engineers have had occasion to observe, owing to the severe treatment which such side motion imposes upon the track. It is commonly known that such cars, when fully loaded, are top-heavy, and that the lateral motion of the bodies of these cars has more to do with throwing track out of alignment than has the behavior of any other class of rolling-stock, not excepting heavy freight locomotives. The side swaying of such cars is also the cause of frequent derailments, especially where the track is not in first-class surface. Not only do such derailments occur on curves, but frequently on tangents, as is vouched for by an engineer of maintenance-of-way of one of the best-known heavy-traffic railway systems of the country. On this road the officials have concluded that the height of steel hopper coal and ore cars is too great for the most satisfactory results in maintenance of track and for safety of train operation.

It is also questionable whether rolling-stock for other kinds of freight, like box cars, should be materially increased in height, as it is entirely possible, and quite probable, that, should more room become available, with certain kinds of freight, they also might be loaded to a point where the condition of top-heaviness would become similar to that described for the high steel hopper cars with open tops. Increase in height of load also increases the pressure with which the car bodies can rest upon their side bearings, when careening takes place, as on rough track, making it more difficult for the trucks to swing into proper position when entering curves or when leaving the same, thereby increasing the resistance to traction and the flange wear against the outer rail of curves, and also the danger of derailment.

Mr. Churchill. CHARLES S. CHURCHILL, M. AM. SOC. C. E., Roanoke, Va.—The numerous increases made by railroad companies during the last ten years in specification live loads for bridges in order to provide for change in design and weight of engines and cars, show the importance of placing the specification loading in as simple a form as possible.

The writer called attention to this point specially during the discussion of a paper on "Train Loadings for Railroad Bridges," by Theodore Cooper, M. Am. Soc. C. E., in 1894,* presenting diagrams drawn up in the same form as those accompanying this discussion, showing thereon the simple and effective form of specification loading then in use on the Norfolk and Western Railroad.

These diagrams, showing equivalent uniform loads per foot which will produce on any given span the same maximum bending moment and shearing force as the given systems of loads and actual engines, bring clearly before us the fact that fractional figures common to engine design should be entirely omitted; and further, that a typical engine with tender is not necessary or even advisable when simplicity and ease of calculation are sought.

Mr. Hodge publishes for the specification loading of the Norfolk and Western Railway, 1903, a loading that was sent out for a few bridges at the beginning of 1903, pending the issuance of the revised standard specifications of the Norfolk and Western Railway which bears the date of July 1st, 1903.

The diagrams, Figs. 7 to 10, show this standard specification loading, also the heaviest engines now in use on the road.

For a still further and closer comparison of this system of loading with other systems, and with actual engines, Figs. 8 and 10 show actual moments and shears.

Calculation is facilitated greatly by the use of an odd number of equal concentrations in the specification loading because the center of gravity of the system coincides with one of the points of concentration. The uniformity of the loads and the regular spacing of them still further simplifies the calculations.

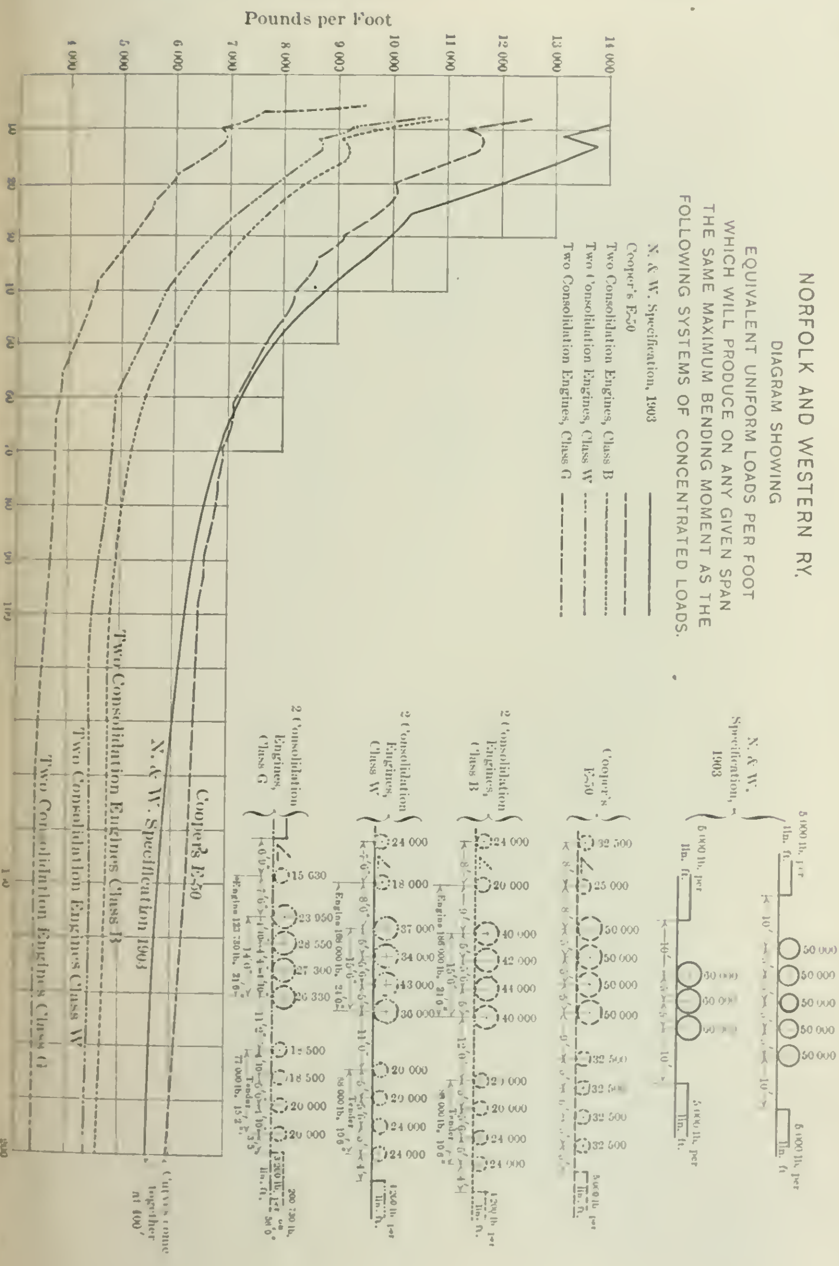
The five-axle concentrations in the specification loading correspond with freight engines, and the three-axled one with heavy passenger engines.

The diagrams are so clear in themselves that the above general statements are almost self-evident, and present an argument for greater uniformity such as Mr. Hodge recommends, also for still greater simplicity.

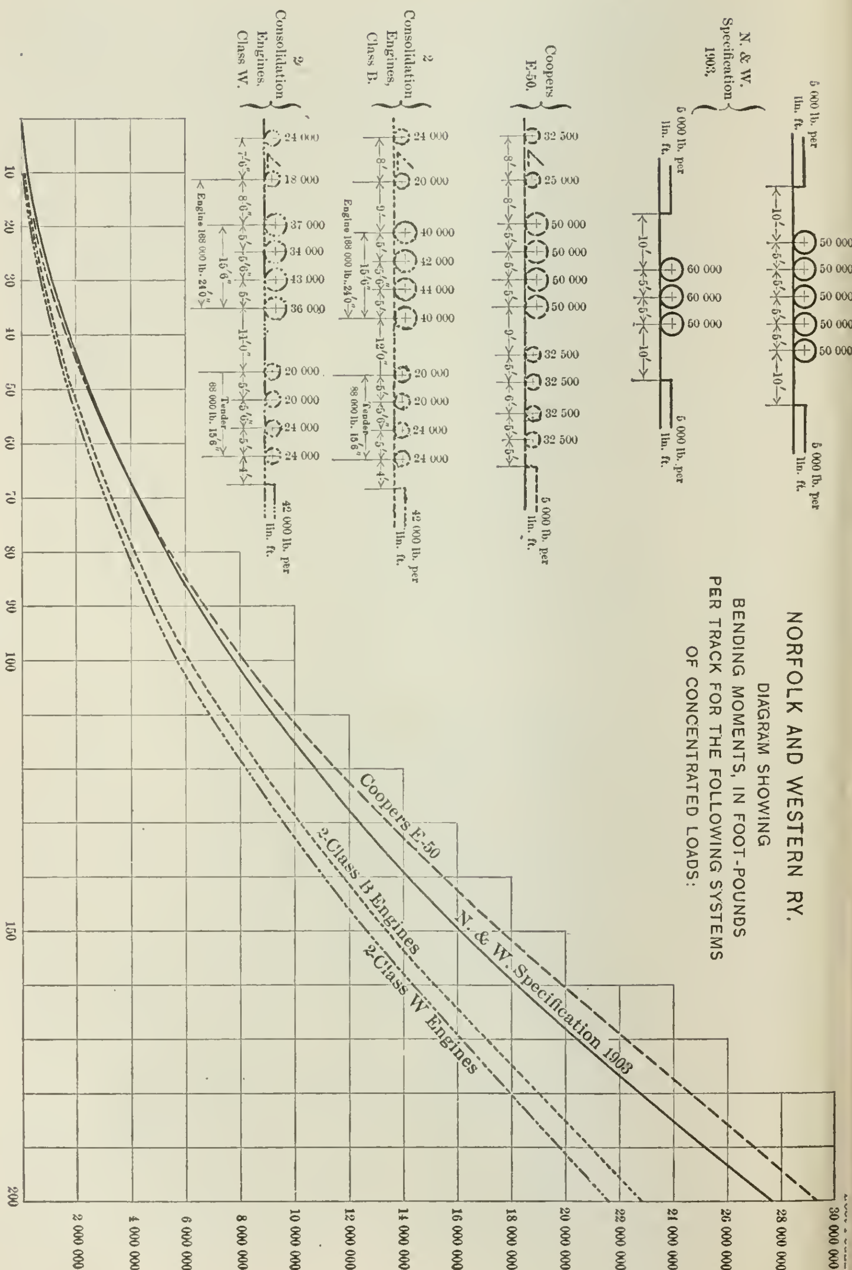
The proposed plan, which amounts to a modified form of typical engine, does not follow necessarily the driver spacing of the actual engine, nor does it give an allotted spacing for the wheels of an

* *Transactions*, Am. Soc. C. E., Vol. XXXI, p. 189.

Mr. Churchill.

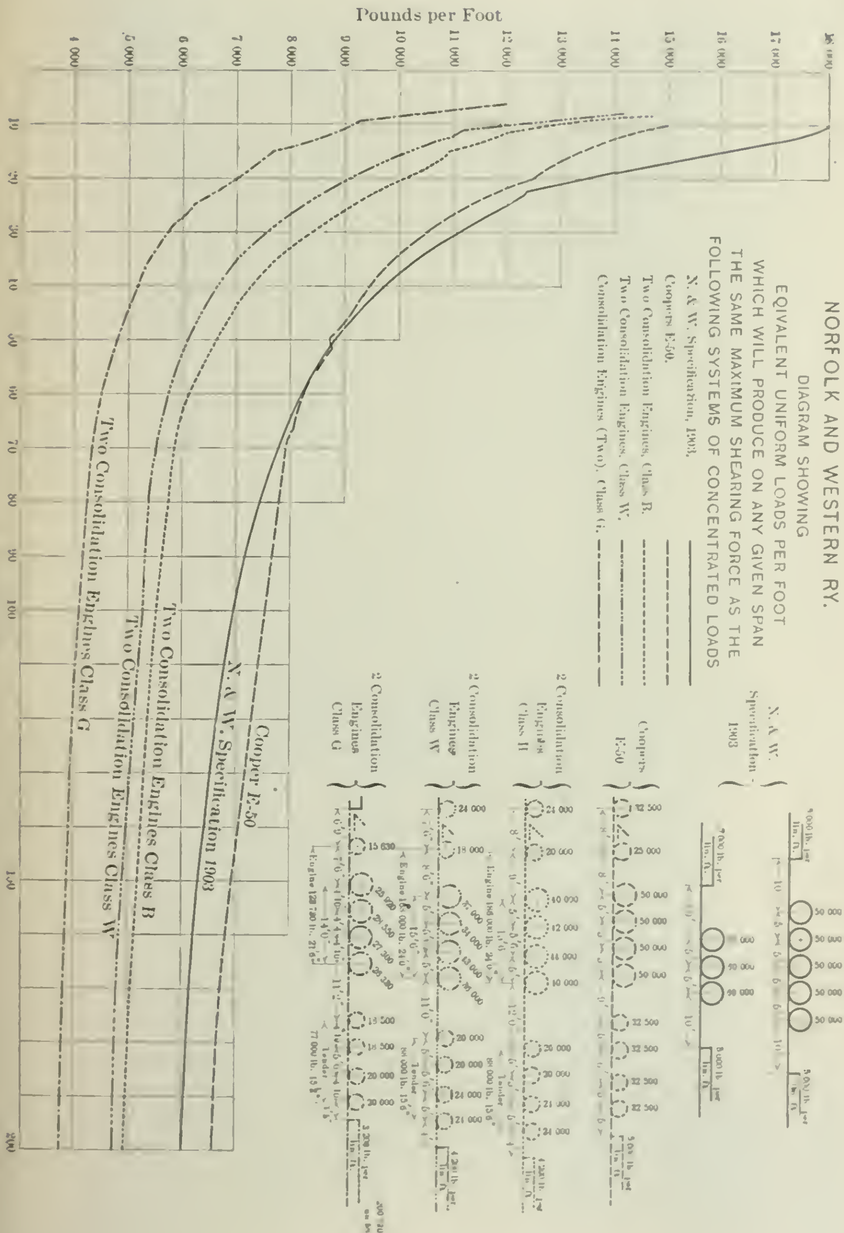


. Churchill.



Span, in Feet
Fig. 8.

Mr. Church.



Span, in Feet
Fig. 9.

Mr. Churchill.

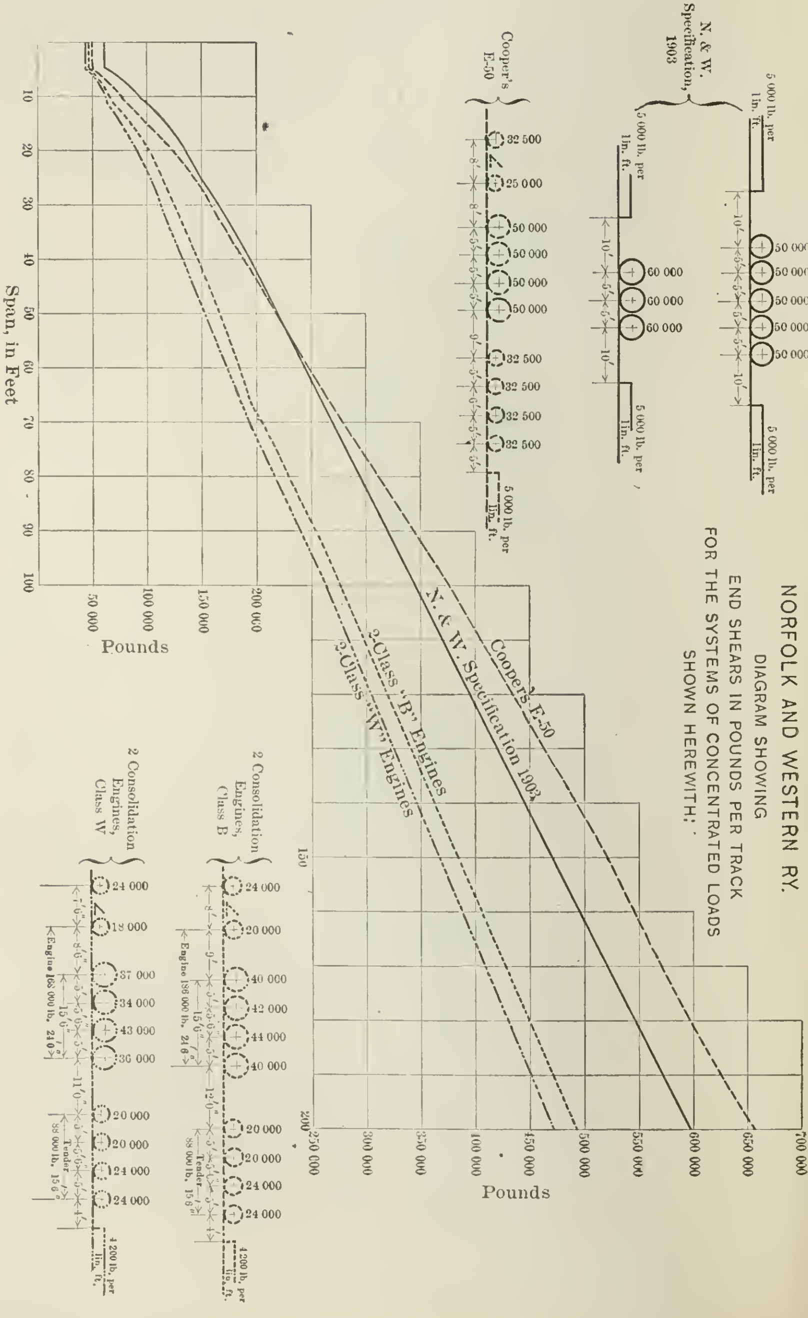


Fig. 10.

actual tender, because the latter is not required. When axle loads of an actual engine and tender are used for live loads of bridges, the tender causes complication in calculating the center of gravity of the entire typical engine, and, as heretofore explained, this can all be avoided. Mr. Churchill.

The speaker presumes that it would be possible to find a single concentrated load, which, in connection with the uniform load, would approximate the same shears and moments as are obtained from the actual engines and trains in service; but, in investigations on the Norfolk and Western Railway, we have found it advisable to have from three to five concentrations to make the effect more closely similar to those actually obtained in service, and while the speaker does not now recall the figures made many years ago on these lines, we had very good arguments for not adopting, at the time, the single concentration. The loadings used in 1891 and 1897 are correctly shown in the diagram on Plates I and II, forming part of the paper under discussion, the speaker has already presented the axle concentrations now in use on the road.

Mr. Reichmann.

ALBERT REICHMANN, M. AM. SOC. C. E., Chicago, Ill.—The character of the permanent improvements on American railroads is rapidly improving from year to year, as the country through which the roads pass becomes more developed. It would seem proper, therefore, in considering bridge loadings, that due consideration should be given to the possibilities of ballasting all steel bridges at some future date.

Some railroads in the West use ballasted bridges quite extensively and with very good results. The increased weight of the structure should be very small, as the ballast materially reduces the inpact of the live load and therefore higher unit stresses may be used.

Mr. Johnson.

J. M. JOHNSON, M. AM. SOC. C. E., Louisville, Ky.—The speaker believes that simplicity would be obtained by a system of uniform loads, with a single concentrate.

This plan has been in use on the Pennsylvania Lines West of Pittsburgh, for some years. and seems to give satisfaction.

For main lines, 5 000 lb. per ft. of track plus an axle load of 50 000 lb. is specified.

For lines having not so heavy traffic, the loadings vary from the above to 3 000 and 30 000 concentrate, as a minimum.

Mr. Robinson.

A. F. ROBINSON, M. AM. SOC. C. E., Chicago, Ill.—In so far as engine loading is concerned, on the Atchison, Topeka and Santa Fé Railway, we simply had to increase that loading to meet the new locomotives which the Company wanted to use. While these so-called Santa Fé types of engines do not quite come up to the heavy grade loading of our bridge specifications, they come very close to it.

Mr. Robinson. Very good results have been obtained with these engines, and the bridges built for our standard loading are thus far behaving satisfactorily under them, but the Company does not expect to run this very heavy power over these bridges any more than is actually necessary. On the divisions of the line where the heaviest engines must run, as fast as the bridges are to be rebuilt they are designed for the heavy grade loading.

It seems to the speaker that, in so far as the actual labor of designing and working stresses in a bridge is concerned, it matters but very little what kind of loading is used, whether it be axle loads or uniform train loads with the various combinations of engine excess. There is also very little difference in the cost of the structures made on the two designs. When it comes to maintaining bridges on a large system, where there are a great many engines of varying sizes and working conditions, one must have a thorough knowledge of working stresses under axle loading of the engines actually in service. In the practice on the Santa Fé System, the engines are divided into different series or classes, beginning with the standard loading of the bridge specifications, and running from that down to the loading of the old Cooper specifications of 1879. Later it was necessary to add a class which was heavier than the standard loading to meet the new conditions imposed by the Santa Fé, Mikado and Decapod types of engines. The heavy grade loading is 50% above the standard, showing 66 000-lb. axle loads for the heavy grade as against 44 000 lb. for the standard loading for the driver axles. The speaker has worked out the end reaction, center moment and floor-beam shear for the standard loading for spans from 4 ft. up to 300 ft., using 25-ft. panels for those above 100 ft. He has also worked out the same for the various classes of engines, but has shown the reactions, moments and beam shears in percentages of those given by standard loading. With these tables, and having a strain sheet showing the live-load stresses separate from the static, it is a very easy matter to tell just what stresses any live loading will produce. When the unit stresses in the floor beams and stringers, in flanges of girders, and in main truss members are run up anywhere from 16 000 to 20 000 lb. per sq. in. net, it always seems to the speaker that one should find as nearly as possible the actual stresses disregarding impact, and that cannot be done with the ordinary uniform loading and engine excess.

In his own work, the speaker has found that after the moment diagram and the end reaction, shear and moment tables were finished for any loading, it is no more work to design the bridge for the wheel loads than for the varying conditions of uniform loading and engine excess. When it comes to the designing, it is very hard to show much difference in the final weight of the bridge as designed

Mr. Robinson. by any one person on two systems of loading. There is a greater difference in the weight of the finished structure, due to the personal equation of the men doing the designing. On the whole, and in view of the fact that, aside from impact, the engine-wheel load stresses give what actually exists as nearly as can be calculated, it is much better to design bridges by the wheel-load method, and it requires no more labor by one method than by the other.

The Atchison, Topeka and Santa Fé Railway System has been doing a great deal of work for some years past in the way of experimenting with ballasted floors on bridges, both with creosoted pile and timber-trestle structures and also for permanent work, and the results have proved unusually satisfactory. C. D. Purdon, M. Am. Soc. C. E., designed the first ballasted deck-trestle bridge for the Santa Fé Company, and the speaker has only carried out what he commenced.

The first metal structures with which the speaker had to do were those for the Joint Track Elevation work in Chicago, and the Los Angeles River Bridge at Los Angeles, Cal. This latter was rather an unusual structure. The stream was crossed almost lengthwise. The skew was something over 34 ft. in a width of 16 ft. 6 in. between centers of trusses. Two 200-ft. spans and two 90-ft. through-girder spans had to be constructed. The track across the girder spans was on a 5° 30′ curve, and the track across the entire bridge was on a 0.6% down grade going west. In addition to this the City authorities of Los Angeles asked for a bridge which would give a pleasing exterior effect, as the structure was nearly in front of the main entrance of the City park known as the Elysian Fields. In addition to the above troubles the distance from base of rail to high-water line was only about 4 ft. 6 in. With this unusual skew, the curve and other conditions, it would not have been advisable to build the structure with an ordinary open deck, as the maintenance of the same would be out of all proportion, and the results of operation would have been unsatisfactory. The ballast floor bridge showing a depth of 3 ft. 3 in. from base of rail to lower clearance was built, and the structure has been in service since 1899, has required nothing in the way of repairs, adjusting or fitting up, and is giving most satisfactory service. The trainmen report that they hardly know when they are on the bridge, and they run just as fast as the location will permit without any reference to it.

As from time to time sufficient reason, from the financial or maintenance standpoint, could be shown, bridges with ballasted floors have been put in on other parts of the system. At Pueblo, Colo., where there was a span with the piers washed out, our engineers decided that a 260-ft. span with an ordinary deep floor would be required, but a 210-ft. span with ballasted floor which

Mr. Robinson. would give as much waterway and cost less money than the 260-ft. span, was substituted. This bridge has been in service some eight months and is giving the very best satisfaction.

We have not undertaken to put in ballasted floors on our metal bridges generally as yet. The design of all structures, however, provides for the addition of ballast. It was thought best not to undertake to ballast everything at once, on account of the high cost. A few bridges on each operating division have been put in, however, as object lessons.

Nearly all metal bridges will cost, for line and surface of track and for fitting up to make them ride properly, from 25 to 40 cents per ft. of track per annum. With ballasted floor, all this work can be done with cheap labor, that is, with section-men, and the result in cost is, we may assume, from 6 to 10 cents per ft. of track per annum. The saving between the two will more than cover the 5% interest on the increased cost, due to the ballast. This does not take into account the perfect riding structure that is obtained from the ballast, the greater safety in case of derailment, the longer life of the structure, or the reduced liability from fire. Before the speaker finishes his work on the Santa Fé System he expects that bridges with ordinary open decks will be the exception rather than the rule.

As to creosoted ballast-deck trestles, the management some years ago made a rule that in renewals, or in replacing temporary structures, a creosoted ballast-deck structure was to be used whenever the estimated cost was not more than $25 per lin. ft. When the cost was more than $25, the so-called permanent structure was built. The results from the use of these ballasted deck trestles have been more than pleasing. When we first commence building them, there were few of the officers on the system who believed in them. To-day there is hardly a man from top to bottom, who is not a thorough believer in the ballast-deck work. Of late, the only query of the operating officers has been: "How long will these creosoted trestles last?" On this score information was obtained from the Louisville and Nashville Railroad which road had been using similar structures for more than 25 years, in which time the Chief Engineer stated that he could not tell what would be the life of these structures, as he had been building them since about 1878 and had not yet taken any out because the material was rotten.

Some two or three years ago, tests to show the deflection of the stringers, or floor, on these ballast-rock structures were commenced by the speaker and some things were found that were a little bit surprising, and others that seemed to carry out the ideas that had been formed. He expects later to show the whole series of diagrams with a study, but, roughly speaking, it was found that when the

stresses in the structures were figured, assuming the certain distribution of the loading, about 9 ft., and, from that figure, deducing the theoretical deflection, about one-half of the figured deflection was found. The deflection of the various pieces making up the deck was essentially uniform for a width of about 9 ft. The speaker has not worked up the results sufficiently as yet to give positive figures. Mr. Robinson.

Referring to Mr. Purdon's remarks it seems to the speaker that he has, for the time being, dropped the matter of vertical throw from counterweights on the drivers at varying speeds. We have engines, the Prairie type, for instance, with 22 500 lb. per wheel normal loading. Under a speed of 70 miles per hour, these engines show something like 13 000 lb. vertical throw from counterweights. The Santa Fé type of engine does not show quite as much, but at 40 miles it is in the neighborhood of 8 000 lb. Now, these matters must be taken into account in calculating. The method of providing for the impact is by the old formula of the live load being double the effect of the static. It may also be said in this connection that our balanced compound engine, Atlantic type, has something like 112 000 lb. on two driver axles. Under a speed of 70 or 75 miles per hour, there is no appreciable vertical disturbance from the counterweights.

There have been a great many girder bridges on the Atchison, Topeka and Santa Fé Railway, which were known ordinarily as shelf-angle girders, where the distance from base of rail to clearance was limited. At the time they were designed we had a floor of 8 by 12-in., or 8 by 14-in., ties resting on the shelf angles riveted to the main girders. The heavy power which has been put into service within the past few years made these floors too light, and the ties gave trouble. In order to overcome the difficulty, as fast as the old timber decks were worn out, they have been replaced by a solid floor and ballast, using 8 by 14-in. timbers and laying them solid, covering same with a course of ballast not less than 6 in. in thickness below the ties. The unit stresses of course are increased somewhat, but the bridges did not seem to deflect as much as they did before the ballast was put on and they are much stiffer laterally. There is one span, some 70 ft. in length, on the Illinois Division, where the track is part of the way on a curve and the rest of the way on a tangent and where trains are always run very fast. For several years one of the girders had a marked lateral bend near mid-span. Under the fast heavy trains, the entire span swung in on the curve, rather than out. The maintenance officers got frightened and eventually a bent was placed under the bridge. Two years ago the old deck was taken out, and replaced with a ballast floor, such as has been mentioned. Since then the bridge has been doing very

Mr. Robinson. good service and shows no undue deflection and no unreasonable lateral swing. It gives every indication of having a good many years of service left in it.

It may be said that, on the new line which the Santa Fé commenced building a year or two ago, known as the Abo Canyon Line, some 80 miles were under contract and laid out. The Abo Canyon had to be crossed a good many times. The track was part of the time on the curve, always on grade, and the piers were frequently skewed. The work was designed with deck structures all the way through and using girder spans up to 100 ft., which are to be erected without the use of false work. These crossings show a height from base of rail to bed of stream varying from 20 ft. up to nearly 150 ft. The girders will rest on concrete piers. The girders are spaced about 10 ft. between centers and are to be covered with a solid floor of 10 by 12-in. section creosoted timbers laid on the flat and topped out with ballast. The deck is 14 ft. wide. On the outside of the deck there is to be a sidewalk 2 ft. wide, with an outside railing 3 ft. high. The contract for the erection of this work was let on the basis of about $1 per ton over the cost of erecting same with false work in the usual manner. This was on the basis of a total of about 3 000 tons.

Any holes or cuts in the timber are to be coated with creosote and the specifications for pile and timber-trestle bridge plans of the Santa Fé System note that wherever the timber has any cuts or the original treated skin of the timber disturbed, whether it be by sawing, adzing or other means, the same must be coated with creosote before assembling the parts, the idea being that this will have a tendency to seal the timber in the same manner as glue. Of course, this has not been used long enough to be absolutely certain that it will do.

Mr. Hodge. Henry W. Hodge, M. Am. Soc. C. E., New York City. (By letter.)—The writer has read with interest the foregoing discussion. The final conclusion of the paper, that any road which expects to carry ordinary freight traffic will be unwise to use a load of less than 50 000 lb. on drivers, followed by 5 000 lb. per lin. ft., seems to be universally concurred in.

If this loading could be generally adopted by leading railroads, it would be a long step in the right direction, and would probably prevent the replacing of bridges, due to overloading, for a considerable period in the future.

The writer fully agrees with the general opinion that this is a sufficient load for usual practice; and again calls attention to the fact that he did not recommend the heaviest loads shown in his paper for universal use, but simply for roads having exceptional traffic or heavy grades necessitating unusually heavy traffic.

He agrees with Mr. Robinson, that it is as easy to compute bridge stresses with a wheel concentration diagram, as it is with a set of equivalent uniform load diagrams, and cannot account for the large amount of work which has been done by our bridge engineers to arrive at equivalent uniform live loads, which are at best but approximate, and which require different tables of equivalents for bending and shear, when the exact stresses are found as readily. Mr. Hodge.

If simplification is desired, let us make it a real simplification, by adopting an arbitrary uniform load with one or two panel excesses, and personally the writer would be glad to see the load mentioned by Mr. Johnson, as used by the Pennsylvania Lines West of Pittsburgh (and as now in use on other lines), adopted for general use, as he believes this loading will cover all the necessities of the actual wheel loads. It is so simple that no tables or diagrams will be necessary, and the bridge designer will be able to concentrate all his energy on the correct proportions and details of his design, instead of wasting so much in refining the stresses down to the last pound.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

INTERNATIONAL ENGINEERING CONGRESS,

1904.

MINING ENGINEERING.

Congress Paper No. 4.

THE OPERATION OF MINES IN FRANCE.

By E. Gruner, Secrétaire du Comité Central des Houillères de France, Paris, France.

Congress Paper No. 5.

MINING ENGINEERING IN THE UNITED STATES.

By E. Gybbon Spilsbury, M. Am. Soc. C. E., New York City, U. S. A.

Discussion of the Subject by

J. A. Ede, La Salle, Ill., U. S. A.

Note.—Figures and Tables in the text are numbered consecutively through the papers and discussion on each subject.

TRANSACTIONS
AMERICAN SOCIETY OF CIVIL ENGINEERS.

INTERNATIONAL ENGINEERING CONGRESS,
1904.

Paper No. 4.

MINING ENGINEERING.

THE OPERATION OF MINES IN FRANCE.

By E. Gruner.*

Translated from the French by
Paul A. Seurot, M. Am. Soc. C. E.

The operation of mines calls for the solution of more varied and more difficult problems from year to year, since coal beds near the surface, easy to explore and to mine, are getting exhausted, and it is becoming necessary to go to great depths, under water-bearing strata or into strata having little or no cohesion, in order to find poor mineral ores or coal beds containing large volumes of fire-damp.

For several centuries, the tools of the miner were very crude; but each decade of the last century has seen wonderful improvements in the mining art, and the last years have been particularly ripe with progress; thanks to the co-operation of all sciences in the study and solution of the problems and difficulties that were met.

These improvements are due, in a large measure, to the daily relations maintained between engineers and mine owners in the

* Secrétaire du Comité Central des Houillères de France.

two continents, to the International Congresses, to the technical literature, to the more and more frequent travels or trips organized by the great mining societies as soon as they hear of some improvement calculated to facilitate the work, to lessen its dangers, or to reduce the cost of mining.

These continual and rapid exchanges of ideas, and the almost simultaneous application of every new process in all mining countries, make it very difficult to give an abstract of the improvements made in any country, especially in view of the fact that engineers of all nationalities have contributed more or less to all of the improvements to be mentioned.

In the following abstract an effort will therefore be made to give a general idea of all improvements made in mining rather than a *résumé* of improvements made in France alone.

Anxiety caused by the exhaustion of certain beds in operation for a long time, and, on the other hand, the hopes raised by the remarkable studies of stratigraphical synthesis made by eminent geologists, have brought to light the question of deep mining.

Two opposing conditions at once present themselves: Proceed. rapidly, and, gather information carefully.

The Raky system, as practiced in Germany, answers the first of these conditions, which is, after all, the consequence of German mining legislation relating to the prerogatives of concessions.

To answer to the well-justified requirements of French laws, and also of many engineers and directors, and in order to furnish precise information, if not all along the borings or test pits, at least through the strata (soft coal, mineral ores, etc.), which will eventually be operated, different boring operations have been made, particularly in France, to bring about important improvements in the methods of quick boring. Now it is possible to pass very rapidly through exhausted beds, to note with precision the changes of the ground and of rocks, to operate methodically through all interesting strata, and to bring from the greatest depths samples almost intact which sometimes may show by their own inclinations the angles of the various strata and furnish information about the important veins.

These improvements in the art of boring enable us not only to discover unknown beds, but also, through the application of the

freezing process, to sink shafts of great depth and of large diameter through blocks of frozen sand, water and rocks, below water courses heretofore considered almost insurmountable obstacles to certain operations.

This process, invented by Poetsch, in Germany, was really not used in practice until some improvements of details were made, particularly by French engineers. These engineers have reached the greatest depths with this process, and the owners of the Belgian Campine Collieries seem to have decided to trust them with the task of reaching coal beds discovered by several borings in 1901 and 1902, at depths far in excess of any reached before.

Will the freezing process be the only practical means of solidifying a vertical cylinder of great height, into which it will be possible, later on, to descend without danger from water and quicksand? MM. Portier and Saclier do not believe so, and they have designed a new method which does not consist in causing the circulation of brine or a refrigerating solution through hermetically closed tubes, but in forcing grout under high pressure through a tube more or less perforated. This grout will penetrate into all the fissures and interstices of the rock, will force back the water permeating the sands and permeable sandstones and will set rapidly. This idea is ingenious; practice will undoubtedly indicate the best methods of making it applicable to the different kinds of soils.

Methods of pushing headings through rock as rapidly as possible and, at the same time, boring test holes ahead in order to safeguard against any possible sudden irruption of water or fire-damp have been obtained through the improvements made in horizontal boring tools and drills.

Every great mining country has contributed to the solution of these problems, and French designers did not lag behind, spurred on as they were by mine owners from Southern France, where beds and layers in operation are so irregular that an opening in a pocket of damp gas or in water-bearing strata would have entailed great disasters, had not the greatest precautions been taken.

Thanks to the possibility of being able to push headings far ahead of the working galleries, work can go on uninterruptedly and rapidly, owing to the great improvements which have been made in drills.

The best motive power to be adopted for these drills, compressed air or electricity, has been the subject of numberless discussions. It seems pretty well established now that the primary importance of insuring entire safety tends to give the preference to compressed air wherever fire-damp may be encountered; electricity does not seem to work well in percussion tools necessary in hard rock.

The irregularity of coal beds in France and the lack of important mining operations of other ores occurring in regular strata, kept public attention away from improvements in apparatus. But the improvements made in cutting machines in the United States by decreasing their weight and size, have permitted their use in some of our collieries. The increase in power of these machines has caused their successful adoption in the horizontal layers of the Lorraine beds. In this particular, French engineers, left somewhat behind at the start, are now coming to the front, and the mechanical cutting and extracting of iron ore seems destined before long to be a very important factor of mining in France.

Several grave accidents caused about twenty years ago by the use of explosives in mines containing fire-damp or a great quantity of coal-dust, led the French Government to appoint two technical committees, the Fire-Damp Committee and the Committee on Explosives, and these, thanks to the knowledge of their members, have thoroughly studied these questions and drawn conclusive solutions.

The names of M. Mallard and of M. H. Le Châtelier are identified with the labors and findings of these committees, with the discoveries made in connection with safe explosives, with the improvements in safety lamps (Marsaut and Fumat types), with the apparatus devised to reveal the presence of fire-damp (fire-damp meter of Le Châtelier, and lamp of Chesneau), and with the precise determination of the conditions under which depots or storage-places of explosives should be installed, whether at the top or at the bottom of the mine.

The decrease of about two-thirds of the deadly accidents due to fire-damp explosions, and, for the last ten years, the disappearance of these catastrophes, which, time and again, had moved public opinion in France, gave to the labors of these Committees the highest and most decisive sanction.

The immunity from danger in fire-damp mines, not only danger from explosion, but also as a factor in the individual work of each miner, depends, in a great measure, upon the lighting power of the lamps used by the men, and its continuousness. The safety lamps of Marsaut and Fumat answer this requirement and also insure safety from fire-damp; the gasoline lamps of Wolf, and the several types of electric lamps give, besides, the advantage of a constant lighting power. In this instance, too, the manufacturers of lamps in all countries have vied with each other in making improvements, and the paper recently published by M. Cuvelette, a mining engineer, in the *Bulletin de l'Industrie Minérale,* on the use of portable electric lamps in the Pas-de-Calais mines, proves that we are, in France, about to reach a very practical solution.

The double question of safety and hygiene has guided the engineers who have studied the problems of ventilation in mines. In this case the work of French engineers has been predominant in theory as well as in practice.

The works of Combes paved the way; the researches of M. Murgue and his learned study on the equivalent orifice have established the theory of ventilation; the ventilators or blowers of Ser, Mortier, and Rateau have made steep grades and depressions possible and increased the circulation of air more easily, and with more safety than with the old fans, which were themselves a marked improvement on the old furnace system.

While the voluminous elevating pumps limited the possibilities of sinking shafts through water-bearing strata, the clearing pumps, with their main connecting rod and counterweight, took up almost every available space, leaving very little room for the cages and elevators, thus limiting the capacity of the shaft and the output. The adoption of underground pumps brought some relief to that condition of affairs, but it is only since the economical transmission of power by electricity has become perfected that it has been possible to solve the problem of underground drainage.

France took part in this study of the drainage question, which, however, was specially studied in Germany: but here again the practical improvements have been the joint work of all countries.

Will it be safe to develop or even to maintain electric traction in mines which are not entirely free from fire-damp? This is a

question over which several engineers are pondering, and which has led them to make a special study of the application of compressed air in mines, in spite of some objections raised in view of the increase of cost and of the difficulties of keeping the piping in order.

This uncertainty relates not only to the question of traction, but, apropos of all the other applications of electricity, to drills, cutting machines, secondary fans and blowers, and to all machinery or apparatus used in places where a sudden expansion, some unexpected occurrence, an atmospheric perturbation, might cause an insufficient or inadequate ventilation, even for a few minutes.

In France, where questions pertaining to safety are carefully scrutinized by the Mining Corps and by the engineers of the mining companies, these studies are pushed forward; and it is only this humanitarian idea of safeguarding the workingman against any possible accident, that explains the delay in the more general adoption of electric power below ground.

It is also this question of safety, from the viewpoint of firedamp and of danger of fires in thick workings, that has made the adoption of methods using full fills and break-ups so general in France. The existence of inexhaustible sand pockets near the mouths of shafts and at the bottom of clearing pumps with a surplus power compared with the normal flow of water from the galleries, led, in Silesia and later in Westphalia, to the adoption of the method of washing materials by means of a stream of water, and, subsequently, getting the sediment by decanting or draining. Before long, this method, so advantageous because of the cohesion of the materials deposited, will be used in several French mines, when, through some variations, it will be applicable to local circumstances, and to the different materials encountered.

For many years the economy of fuel in a coal mine seemed quite secondary, and the simplicity of boilers and engines was a unique question.

It is quite different now, and, as a proof, attention may be called to the utilization in coke ovens of lost heat and gases, and afterwards of all the by-products which were uselessly consumed in the smoke stack, or volatilized, when they might have been transformed by condensation, and used by agriculturists and manufacturers. This question of recuperating ovens has been first of all studied in

France; with it is identified the name of Carvès of Saint-Etienne. Several improvements were made later on in Germany; although it is mostly to French engineers that the installation of transforming or recuperating apparatus in many countries has been entrusted.

The boilers, too, have been improved, and for many years the question of applying condensation to mining apparatus has been carefully investigated. Here again will be found the name of a young French inventor, M. Rateau, Mining Engineer, who, while studying the applications of the turbine, after designing the ingenious fan or blower known as the Rateau Fan, and after designing several rotary electric pumps specially adapted to the sinking of shafts, the working area of which they do not obstruct, proposed to store the lost power of the extracting machinery (as well as of all machinery), and afterwards to utilize it in running a turbine, working under low pressure. This idea is new; the applications are yet scarce, but they seem in a fair way to increase rapidly, because of the simplicity of the installations and of the economies they realize.

The difficulty of securing a sufficient supply of timber for frames and lagging, the drop in price of medium steel standard shapes, and also, as always, the constant endeavor in France to improve the conditions of safety of the work, have caused not only trials of steel frames, but their current use in headings, as well as in break-ups. These applications are pushed forward and seem to fulfill all the expectations of the engineers who promoted them, particularly in the mines of Bruay, Marles, and Courrières.

In order to avoid any loss due to the abandonment of beds, which it will be impossible to re-enter after the layers already mined will have caved in; the desire to compete with some other mines which can be operated under more favorable conditions; the obligation to free coal sent to great distances of all useless cost of transportation; all these various reasons have led to the installation of machinery, the duty of which is to purify the coal, first, by sorting, then, by washing; and also of briquette-making machinery, utilizing by agglomeration, all dust, crushed or powdered debris, after washing or without washing.

Constant improvements are made in the manufacture of this machinery, which is sometimes very simple, as at Saint-Etienne.

and sometimes very complicated and expensive as in Germany. French manufacturers have endeavored to obtain simple solutions, better adapted to average needs than the German installations, which are chiefly adapted to large production and to very large plants, as in Westphalia and Silesia.

The development of collieries has caused the concentration of large working communities in regions sometimes thinly populated.

The coal mining companies have endeavored to furnish to the workmen and their families, sanitary dwellings with gardens, schools, churches and hospitals; and for this purpose the coal companies of the Nord and of the Pas-de-Calais have made great sacrifices.

Before the time when public authorities officially took up the question of accidents to workingmen, of medical help and of pensions, the collieries had all provided their workmen with reliefs and pensions in case of accidents, with doctor's care and medicines for themselves and families in case of sickness, and with pensions or annuities when age made all work impossible.

From the social viewpoint as well as from a technical viewpoint, it is certain that the men at the head of the French collieries, united for the study and for the defense of their common interests in the Central Committee, have led in the path of progress and improvement, and are still anxious to know all the latest improvements, in order to apply them to the special conditions in France.

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MINING ENGINEERING.

IN THE UNITED STATES.

By E. Gybbon Spilsbury, M. Am. Soc. C. E.

A complete history of all the advances and improvements in Mining Engineering and Metallurgy during the last decade would consume many times the space which has been allotted for this paper, in which, therefore, attention will be given to only a few of the most prominent and far-reaching improvements which have been introduced in the United States.

The science of Mining Engineering is a composite one, and, in mining parlance, might be termed a "concentrate" of all the other branches of the profession. It requires Civil, Mechanical and Electrical Engineering plus chemistry, geology and metallurgy, to make it successful. This being the case, therefore, it becomes a difficult problem, in describing the advances of Mining Engineering, not to encroach on similar advances made in Civil, Mechanical and Electrical Engineering fields during the same period.

Mining Engineering may be classed under the following heads:

First.—Mining proper:

a.—The methods of developing ore bodies;

b.—Appliances for extraction of the ore;

c.—Devices for the control of underground waters and ventilation.

Second.—Primary treatment of the materials extracted:

a.—Mechanical preparation;

b.—Concentration;

c.—Local transportation.

Third.—Final treatment and reduction:

a.—Final utilization of the extracted mineral;

b.—Metallurgical treatment of the minerals for the extraction of the contained metals;

c.—Electrical treatment for final separation and refining of the composite metals produced under Heading *b.*

All mine products are not subject to all of these many processes, but all mine products are treated by one or more of them.

During the past decade there have been but few startling or revolutionary inventions introduced into Mining Engineering, but still the advances made during that period have certainly been equal to, if not greater than, during any similar period. The growth of these improvements has been so gradual that it is hard always to appreciate how great they have been.

While in the older European countries the tendencies of the improvements have been principally in the direction of closer extraction, greater purity of materials extracted, and economics on those lines, the trend in this country has been toward greater production and lower cost of product. Our wage rates being so much higher than those of Europe, our incentive to introduce labor-saving appliances is naturally much greater, and we must offset the higher wages by increased production per wage unit.

One of the most distinctive characteristics of American Mining Engineering is the manner in which the difficulties or restrictions met with in different districts have been handled and overcome without regard to tradition or precedent.

When the enormous ore bodies of high-grade ores in the Comstock Lode were discovered, presenting difficulties almost insurmountable to the then known methods of stoping by means of "stulls" and props, it took our miners but a short time to evolve the American system of "square sets" with filling, which has so

successfully enabled us to extract those large ore bodies to depths of nearly a mile.

When, later, the great contact deposits of silver lead ores in the Leadville District were discovered, only a slight modification of this system was necessary to meet the requirements.

Almost within the last decade, the discovery of the enormous iron ore deposits of the Mesabi Range in Wisconsin has called forth a novel system of mining, which is so pre-eminently American as to deserve a special description.

For some of the following details the writer is indebted to the able papers* on this subject by Messrs. C. E. Bailey, James E. Jopling and Professor F. W. Denton.

The iron ore deposits of the Mesabi and, to a certain extent, those of the Vermilion Range in Minnesota occur in a blanket formation, overlying conformably the quartzites while imbedded in the taconites which form the walls of the deposits.

These deposits are beds of great area varying in width from 200 to 3 000 ft., and sometimes extending several miles in length. These beds, while on their surface nearly horizontal, have a well-defined general southerly dip of 4 to 5 per cent. The iron ore itself does not often reach the present surface but is overlaid with glacial drift varying in thickness from 2 to 200 ft. This overburden is a stiff clay mixed with granite boulders near the surface, changing to fine sand and gravel in contact with the ore.

The depth of the deposits varies considerably, reaching in some few mines 500 ft. and over, but the average may be taken as from 45 to 60 ft. The ore is a comparatively soft hematite, although occasionally hard ribs are found in it.

It soon became evident, after the discovery of these deposits, that those with a light overburden could be worked best by open cast methods, after stripping off the surface gravels, and, in the beginning, this was the general method adopted.

The great area required to be uncovered and the average depth of more than 25 ft. of overburden soon called the steam shovel into use for the stripping work, and, while the first ones used for this purpose were too light for the duty, the development of this machine

* *Transactions*, Am. Inst. Min. Engrs., Vol. XXVII, pp. 529, 541 and 344

has kept pace with the requirements, so that to-day there are steam shovels at work in this district, capable of excavating over 1800 tons of ore per day, at a cost for operating not exceeding $65.

There is, of course, a limit to the depth at which the steam shovel work can be carried with economy, and the Mining Engineer was forced to prepare some other method to supersede the shovel, when this depth was reached.

The ordinary method of stoping with square sets and filling would, in this case, be too expensive. An entirely novel system was, therefore, evolved, which is locally known as the "milling" system. To carry out this method, a main shaft is sunk outside the ore deposit, in solid rock, to a depth equal to the bottom of the ore bed, and drifts are run out at this bottom level to all parts of the deposit; at regular intervals upraises are made to the surface. The ore is then broken down through these raises and loaded from shutes into cars at the bottom, which are hauled by horse or electric power to the main shaft and hoisted to the surface. In some of the mines the steam shovels are used to excavate and dump the ore into the raises.

Still another system has been developed, known as the "caving" system, which does away altogether with the necessity of preliminary stripping off the surface overburden of gravel, sand or clay. In this case, also, a main shaft is sunk in the wall rocks, and, from the bottom, drifts are run into the ore body. Intermediate levels—called sub-levels—are run at distances of about 20 ft. These levels are connected by upraises to the top of the ore body. The ore is then taken out in slices, of such width as will sustain the roof temporarily while working. The ore is shovelled into the raises and loaded into cars, as in the "milling" system. As each slice is excavated, the overburden of rock and gravel is allowed to cave into the opening. This dead material packs tight enough to allow the removal of the next slice. The operation is repeated until the whole top of the ore body is removed and dumped through the raises to the floor of the first sub-level.

As each slice of ore is removed, the floor of the first level is covered with planking to prevent the dirt from mixing with the

ore, and, also, to help in forming a temporary roof when the raises from the next sub-level begin to slice off the next block of ore. This system has been found very economical, and, practically, all the ore is saved.

The writer has gone somewhat into detail in this description, as he considers that this is an evolution, entirely American, of one of the most difficult problems for the cheap handling of large bodies of low-priced material.

Another example of an entirely novel method of mining is also worthy of mention here. This is the method now being so successfully used in extracting sulphur from the Calcasieu deposits in Louisiana.

In order to understand fully the difficulties which it has been necessary to overcome, a short description of the facts relating to the deposit is necessary.

The sulphur deposit consists of an immense bed of pure sulphur in gypsum, extending over a known area of more than a mile square, and varying in thickness from 60 to 150 ft. This deposit was first discovered in boring for oil some forty years ago, at a depth of 700 ft. A large number of bore-holes were put down to determine the limit of its area, and operations were commenced at once to attack the deposit by a shaft.

Owing to the character of the ground overlying the deposit, which consists of alluvial deposits, quicksands, gravels and clays, it was found impossible to sink a shaft by ordinary means, and the services of a Belgian engineer, Mr. Julian Deby, were engaged to introduce the then novel system of "Kindt-Chaudron" for sinking.

Notwithstanding the most careful work on his part, the shaft was lost when only a little over 100 ft. had been sunk. The work was then abandoned for many years. In 1892, however, a new company was formed, and another effort was made to sink a cast-iron caisson shaft through the quicksands to the limestone formation which was supposed to cover the deposit. This effort also resulted in an absolute failure. Before undertaking the sinking of a third shaft, the company wisely determined to investigate carefully the whole problem by a series of new borings, and not to rely.

as heretofore, on the records of the old bore-holes. They soon found that the supposed water-tight stratum of compact limestone, reported as overlying the sulphur bed, did not exist, and that no shaft could be made water-tight unless a very large amount of the sulphur deposit itself was left as a permanent roof, thus greatly diminishing the available amount of ore to be obtained.

These new drill-holes further established the fact of the existence, in large quantities, of poisonous gases, chiefly sulphureted hydrogen, in the fissures of the sulphur deposit, so that, even if a shaft could have been successfully sunk and made reasonably water-tight, the extraction of the sulphur would have been a very dangerous operation, if not actually impossible.

While these problems were under discussion, Mr. Herman Frasch, Chief Chemist for the Standard Oil Company, who had been giving the matter a great deal of attention, came forward with the bold proposition to melt the sulphur underground, by means of steam or superheated water, and pump the molten sulphur to the surface.

As may be well imagined, the proposal of so novel and so thoroughly radical a system was met at first with considerable scepticism, not only on the part of the directors, but also of the expert engineers called in consultation. Mr. Frasch had, however, studied out the proposition so thoroughly that he was soon able to convince his opponents of the feasibility of the scheme, and it was determined to make a trial of his system, which has now resulted in such a remarkable success as to revolutionize practically the sulphur trade of the world.

The present success, however, was not attained from the outset, but is the result of the most indefatigable courage on the part of Mr. Frasch in meeting and overcoming the many difficulties of detail encountered as the work progressed.

In brief, the system, as now perfected and in use, consists in boring an 8-in. hole from the surface to the lower floor of the deposit of sulphur. This hole is cased in the ordinary manner. Through this casing is passed a 6-in. pipe, in which is a 3-in. and, finally, a 1-in., pipe. Through the 6-in. pipe, hot water,

under a pressure of 60 lb. to the square inch, is passed down into the sulphur deposit. The sulphur soon begins to melt and is forced up through the 3-in. pipe by compressed air passing through the 1-in. pipe.

The description appears simple enough, but the working out of the details necessary to ensure the permanent success now achieved has required many years of intelligent work.

At the present time, the pumping averages from 350 to 400 tons per day from each well. The production for July, 1904, even reached 16 000 tons, and could be increased at will to double that quantity. The total product that each bore-hole will produce cannot of course be determined; some give out and stop flowing after a yield of 3 000 to 4 000 tons, while others have yielded over 23 000 tons before being considered exhausted.

The sulphur is allowed to flow into immense reservoirs, about 70 ft. square and 12 ft. deep, and is cooled in 10-in. layers, so as to facilitate the breaking up and loading on cars.

The quality of the sulphur thus obtained is far better than any of the Sicilian sulphur brought to the United States. It is put on the market under a guarantee of 99.50% pure sulphur, although it really analyzes 99.9 per cent. Already, the company is producing more than this country will consume, and in the summer of 1904, several cargoes, of 3 000 tons each, were sent to Europe.

The cost of production by this system is so ridiculously low that no possible competition can ever be feared from points where ordinary mining methods are used.

Regarding the question of the permanency of this cheap production and the extent of the deposit, it can only be said that so far the limits of the deposit have not been found, while a fair calculation of the quantity of available sulphur in the area already developed by boring shows it to be over 40 000 000 tons.

In mining for gold in gravel deposits, during the last few years, the use of the dredge has grown from a mere experiment to a well-established and largely developed system.

Adopted primarily for the purpose of recovering the gold deposited in the bottoms of rivers, the use of the dredge has been

successfully extended to gravel benches high up in mountains, and in places where there would have been too little water available for the usual hydraulic methods. All that is necessary is to excavate a tank or pond sufficiently large in which to float the dredge, which then goes on digging itself into the banks, while piling up the débris or tailings behind it.

The history of the wonderful development that has been made in the construction of these dredges belongs to the department of Mechanical Engineering, but the requirements which caused this development were made by the Mining Engineer.

The present dredge for mining work must have a capacity to handle from 2 000 to 2 500 cu. yd. per day, digging the material either from the bottom of the river or from the benches surrounding it; passing it through the washing drums, separating the fine sands and treating them on tables and riffles, and, finally, removing all the material once more and stacking it in piles behind it; all this to be done at such a low cost as will return a profit to the operator when the entire yield in gold may not exceed 30 cents per cu. yd. The rapid increase in the number of these dredges is proof of the success of the method.

In the matter of proper ventilation of American mines, not much improvement can be noted outside the coal regions, where a more rigid inspection necessitates improved methods. The old Guibal fans are now generally being replaced with more efficient high-speed, electrically-driven blowers, but, with very few exceptions, do we find the same care and study given to this question as has been done in European and, especially, in French collieries.

On the other hand, in the manner of rapid handling of material underground and hoisting it to the surface, wonderful advances have been made. The use of both electricity and compressed air for underground haulage has become most general, and hoisting appliances having daily capacities up to 4 000 tons have been installed.

The advances in pumping machinery and the handling of underground waters have also kept pace with the other improvements. The old-fashioned and wasteful steam pumps have been replaced by the more economical compound and even triple-expansion, under-

ground pumping engines, which even now are threatened with displacement by the new high-speed turbine pumps, generally driven by electricity.

The simplicity and compactness of this new turbine pump, added to its effectiveness under high pressures, are going to be great boons to the mine operator. It eliminates all steam and exhaust pipes from the shaft, requires little or no attention while running, and, as it can be worked against heads of 1 000 ft. and over with almost as much economy as on lower lifts, there are very few mines where relays will be found requisite.

In the mechanical preparation of the crude minerals extracted, for market uses, the principal advances made during the past decade have been in the increased capacity of the appliances used, and in the mechanical devices for handling the material through the different operations.

Probably one of the greatest factors for cheap handling has been the introduction of the belt conveyor or carrier, notably that of the Robins type, which, with its automatic distributing appliances, enables, not only the conveyance of materials throughout a mill, but also the discharge of them at any given number of points, in any given quantities, without the intervention of hand labor of any description.

The appliances for crushing and pulverizing the material have been improved to a vast extent, not only in capacity, but also in endurance. The gyratory type of crusher, having a capacity of 250 to 300 tons per day, is fast supplanting the breaker of the old Blake type, while the high-speed, large diameter and narrow-faced rolls have brought about a great increase of production at a much lower cost for power.

The concentration of the mineral products from their gangues by wet methods shows little improvement during the past few years, except that better results are generally being obtained, as greater care is taken in a proper sizing of the material treated.

On the other hand, the Wetherell discovery of the possibility of magnetically separating minerals which were supposed previously to be non-magnetic has resulted in the development of a number of

magnetic separators, and has given a great impetus to this branch of concentration.

It has been found that most minerals containing even the smallest traces of iron in their composition are or can be rendered magnetic to a certain degree under given conditions of current and temperature.

By passing the finely divided material, therefore, successively into the field of a series of magnetic currents of different intensity and volume, a separation can be obtained of nearly all the component minerals in the material. So far, the most successful field for this system has been in the separation of zinc ores from their gangue and other materials. Probably the largest plant using this system is that of the New Jersey Zinc Company, at Franklin, N. J., where they crush and concentrate 2 000 tons per day of zinc and franklinite ores.

A new method of ore concentration has been developed within the last few years, and has considerable merit. It contemplates the use of oil, or oleaginous compounds, as well as water, for the separation of the metallic particles from those which are non-metallic. Several systems have already been introduced, all claiming improvements over the original Elmore system. The improvements consist (with possibly one exception) in the methods of handling and treatment, rather than in the use of the oil. The oil concentration is based on what appears to be a curious fact, that the surfaces of metallic minerals seem to have the faculty of coating themselves with a highly tenacious film of oil, while the non-metallic substances lack this power. The result of this is, that if the mass, having first been thoroughly treated with oil, is then plunged into water, the free oil rises at once to the surface and brings with it all the metallic substances, while those which have freed themselves from the oil sink to the bottom. It will be seen, therefore, that this is an entire reversal of all previous systems of ore concentration based on the separation of the particles of different minerals according to their specific gravities.

The slimes or pulp with the oil are then conveyed to a centrifugal machine, where the bulk of the oil is separated and again made ready for further use.

The latest adaptation of this system is known as the Schwartz system. It contemplates the use of a fatty non-fluid compound instead of ordinary oil. The ore is first mixed with this compound at a temperature high enough to maintain its fluidity; then water is introduced, and the mass is kept triturated and stirred, while fresh water is constantly added; as the compound is cooled off, it coagulates and encloses with it the metallic particles, while the gangue is washed away with the waste water. The fatty compound is then recovered from the metallic particles by first heating to render it fluid, and then passing the mass through a centrifugal separator.

While, doubtless, there is a great future for this system of oil concentration, it is never likely to come into general use for all classes of ore. The restrictions will be found to be:

First.—That it will only separate the metallic from the non-metallic substances, and, therefore, in the case of composite ores, the final product would require working by some other process.

Second.—The cost for oil and the extra handling required, first for mixing and then recovering the compound, only make it available for high-grade ores, or material of considerable value.

Besides these objections there is still a further one, caused by the policy of the present owners of the different patents, who refuse to sell their appliances out and out at a reasonable profit, preferring to charge a tonnage royalty on all the ore treated. The really legitimate extension of oil concentration will, therefore, not be attained until after the original ground patents have expired.

The next great question in Mining Engineering is that relating to the generation of power. This is one of the most difficult problems the Mining Engineer has to contend with.

Generally situated away from machine-shop centers, confronted with difficult transportation, with poor fuel and water, or even entire lack of one or both, this power question is a hard one to solve. The modern refinements in the way of condensing, compounding or automatic firing, etc., are seldom procurable in the average mining camp. The rule, rather than the exception, gives the miner a high price for a poor quality of fuel, coupled with a short supply of

water, and that of poor quality also. Under these conditions it is not surprising that the general steam engine practice in mining camps has shown but little improvement.

On the other hand, the development of the use of the gas engine for mining purposes has been very marked. This has been greatly aided by the improvements and economics in gas generation made by the Loomis-Pettibone Company. It is remarkable that, even in places where water is available all the year round, it is now found more economical to generate electrical power for distribution through a mining establishment by means of producer gas engines, rather than by compound condensing steam engines.

The final treatment of the material mined for the extraction of the metal contents, while not always carried on by the Mining Engineer himself, belongs properly to the field of Mining Engineering. The wonderful developments which have been made in this field during the last decade are so numerous that it would take fifty papers of the permitted length of this one to attempt even their description.

The improvements in the blast furnace for the production of pig iron have been so radical as almost to revolutionize the process. Twenty years ago a furnace putting out 200 tons of pig iron per day was a phenomenon; to-day, it would be considered as behind the times, and nearly double that quantity is not thought extraordinary. This has been made possible by the introduction of automatic charging machines and improved blowing machinery. The Mechanical Engineer has then stepped in, and his improvements seem to be legion.

The metallurgy of all other ores has advanced fully as much as that of iron and steel. In the reduction of copper ores, the method of pyritic smelting has resulted in so much saving in fuel consumption that ores containing as low as 2% of copper can now be made profitable in some sections of the United States.

The Bessemerizing of copper matte for the production of blister copper, which was originally invented by a French engineer, M. Manhes, has been so improved by American practice that it now supersedes all other methods.

Even in the treatment of gold ores, the improvements made in the last few years to the cyanide processes in this country, show remarkable advances. The substitution of the stirring and decantation system for that of filtration has not only resulted in extending the uses of the process to gold ores having a clayey or Talcose gangue, but has introduced a method of intensely fine grinding for all ores, which permits of a more perfect extraction of the gold within a shorter period of time than was possible under the percolation system. This fine grinding has also extended the scope of this process to the working of many silver ores, enabling an extraction up to 80% of the silver values of such ores.

Finally, the electrolytic processes for the separation and refining of the metallic alloys produced in the smelting process have, within the decade, been brought to a state of perfection hardly dreamt of at their inception. Not only is copper electrically separated from gold and silver, but nickel and cobalt are also refined by the same method.

Lately also in some of our Western mines, an electrolytic process has been perfected for the economic separation of lead from antimony and bismuth with gold and silver. This process deposits each of the metals separately in its pure state, requiring only a remelting of the slimes resulting from each process into new anode plates, to be treated again under somewhat different conditions as to current and electrolyte.

These represent really only a few of the infinite number of improvements and advances being made in Mining Engineering. The conditions a Mining Engineer has to confront are different from those generally encountered by his confrères of the other branches of the profession. There are no fixed laws for the deposition of minerals, or for the manner in which they shall be exploited, and no rules or formulas from which to calculate, with any certainty, the strains to which the material he may have to use in his underground work are likely to be subjected. Generally he is away from centers where he can confer with others as to his work, and, therefore, the improvements he may make, while of great importance and value, are often only so locally, and, consequently, are not recorded or made public.

On the whole, however, the sum of these difficuties successfully overcome has resulted in a most general advance in the methods of Mining Engineering, has lessened the dangers to human life, lowered the cost of production and greatly increased the mineral resources of the United States.

TRANSACTIONS
AMERICAN SOCIETY OF CIVIL ENGINEERS.

INTERNATIONAL ENGINEERING CONGRESS, 1904.

DISCUSSION ON MINING ENGINEERING.

BY J. A. EDE, ESQ.

J. A. EDE, ESQ., La Salle, Ill.—No greater progress has been made anywhere than in the status of the mining engineer himself. Some forty years ago a friend of the speaker engaged an old Cornish captain to examine for him a tin mine, in Cornwall, England. The report was a verbal one, and the synopsis was about as follows: Mr. Ede.

"Mr. Tredinnick if ye sink deep enuff, and dreev far enuff, if its there ell find it, if not ye wont."

The great progress which has taken place in the scientific, mechanical, and other departments of mining, is indicated by the difference between the old pioneer referred to, groping in darkness and uncertainty, and the mining engineer of to-day, whose every path is lighted by the headlight of science and art.

One of the most conspicuous signs of the times is the steady advance of investigation along lines supposed to be impenetrable. The speaker refers to the investigations of such men as Raymond, Van Hise, Posepny, Rickard and others, who are to-day wrenching from Nature the secrets of her origin; demanding and forcing from her a complete history of her past methods of secreting her treasures.

One of the greatest benefits to the mining engineer is the aid that enables him to form some rational conception of the genesis of the ore or other material he has to examine. When he knows something of the precedent conditions, he has been furnished with a

Mr. Ede. premise that will enable him to arrive at a conclusion as to its prospective value.

The progress made in the sampling of mines must also be placed well to the front; for it is the method of determining the intrinsic value of a mine, and is, to-day, the first evidence brought forward.

The progress in timbering and concentration referred to reminds the speaker of the forest of timber at the Copper Queen, Bisbee, Ariz., which he has traversed, and that he was subsequently informed that the waste from the stalls had been removed three times for treatment, the period between each time representing the march of progress; the waste of one time becoming the profitable "values" of another.

In Old Mexico, in the Santa Eulalia mining camp, situated within a few miles of Chihuahua, the past and the present can to-day been seen side by side. A few months ago the speaker visited an old Spanish mine, and it took him half a day to go down its winding labyrinth of descending tortuous excavation. From this depth, about 1 000 ft., the miner plods every day up his chicken ladder, carrying on his back the product of his labor. Not far distant is the Potosi Mine, with its improved American machinery, hoisting from 200 to 500 tons of ore per day.

In reviewing the progress made, we are impressed with the truth of the following facts:

First.—That the mining engineer of to-day having a better knowledge of his subject can with more confidence pass judgment on the quality of that which he is called upon to examine, whether the proposition belongs to the speculative, progressive or remunerative class of mining. This knowledge also helps him specifically in determining the value of speculative investigations.

Second.—That the improved method and practice of sampling places him in a position to gauge, with a greater degree of precision, the intrinsic value of all mine reserves; and to arrive at a more accurate commercial estimate of the same.

Third.—That the improved system of mining—the adoption of electrical, mechanical, chemical and other scientific aids for the extraction, transportation, concentration, and conversion of the raw material to the finished product, has so reduced the cost of mining that he can now save that which was lost, and make profitable that which was formerly unprofitable.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852

TRANSACTIONS.

INTERNATIONAL ENGINEERING CONGRESS,

1904.

HARBORS.

Congress Paper No. 6.

ISLAND HARBOURS AND THE ACCUMULATIONS OF MATERIAL CAUSED BY DETACHED WORKS.

By P. Vedel, Chief Engineer, Harbor of Aarhus, Denmark.

Congress Paper No. 7.

HARBOURS OF GREAT BRITAIN.

By William Matthews, C. M. G., M. Inst. C. E., London, England.

Congress Paper No. 8.

HARBOUR DEVELOPMENT IN HOLLAND.

By H. Wortman, Engr., Royal Corps of Waterstaat, Amsterdam, The Netherlands.

Congress Paper No. 9.

MARITIME PORTS OF FRANCE.

By Baron E. T. Quinette de Rochemont, M. Am. Soc. C. E., Inspecteur Général des Ponts et Chaussées, Paris, France.

Congress Paper No. 10.

THE PREPARATION AND USE OF CONCRETE BLOCKS FOR HARBOUR WORKS.

By I. Hiroi, C. E., Tokio, Japan.

Congress Paper No. 11.

CONCRETE BLOCKS AT OSAKA HARBOUR WORKS, JAPAN.

By S. Shima, C. E., Osaka, Japan.

Congress Paper No. 12.

HARBORS ON LAKES ERIE AND ONTARIO.

By Dan C. Kingman, Maj., Corps of Engrs., U. S. A.

Congress Paper No. 13.

HARBORS ON LAKE SUPERIOR, PARTICULARLY DULUTH-SUPERIOR HARBOR.

By David Du B. Gaillard, Maj., Corps of Engrs., U. S. A.

Congress Paper No. 14.

SEACOAST HARBORS IN THE UNITED STATES.

By Cassius E. Gillette, Maj., Corps of Engrs., U. S. A.

Congress Paper No. 15.

THE DELAWARE, SANDY BAY AND SAN PEDRO BREAKWATERS.

By C. H. McKinstry, M. Am. Soc. C. E., Capt., Corps of Engrs., U. S. A.

Discussion of the Subject by

L. J. Le Conte, Oakland, Cal., U. S. A.
Lewis M. Haupt, Philadelphia, Pa., U. S. A.
W. Henry Hunter, London, England.
P. W. Meik, London, England.
H. H. Wadsworth, Superior, Wis., U. S. A.
John H. Darling, Duluth, Minn., U. S. A.
J. L. Van Ornum, St. Louis, Mo., U. S. A.
Clarence Coleman, Duluth, Minn., U. S. A.
A. E. Carey, London, England.
E. L. Corthell, New York City, U. S. A.
W. Matthews, London, England.
P. Vedel, Aarhus, Denmark.
Cassius E. Gillette, San Francisco, Cal., U. S. A.

Note.—Figures and Tables in the text are numbered consecutively through the papers and discussion on each subject.

TRANSACTIONS

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INTERNATIONAL ENGINEERING CONGRESS,

1904.

Paper No. 6.

HARBORS.

ISLAND HARBORS AND THE ACCUMULATIONS OF MATERIAL CAUSED BY DETACHED WORKS.

By P. Vedel.*

On coasts subject to littoral drift, any solid structure projecting across the beach will cause an accumulation of material. If the structure is a jetty for the protection of a harbor, and the supply of drift is not exhausted, the accumulation, by advancing outward gradually and finally rounding the outer jetty-head, imperils the life of the harbor and in time destroys it by closing up its entrance.

That such accumulation may be diminished, if not entirely avoided, by building harbors as detached works connected with land by open viaducts, is not a modern discovery. When enclosing their ports at Ostia, Tarentum, Pozzuoli, Miscnus, etc., by moles with arched openings, the Romans seem to have had in view not only to economize material and provide covered berths for minor vessels but to let the waves consume their energy and leave a free passage for the current, lest a deposit should be formed. About sixty years ago, Cresy, in his "Encyclopædia of Civil Engineering," considering

* Chief Engineer of the Harbor of Aarhus, Denmark.

that "to form a perfect establishment to receive vessels at all times, it should be at a distance from the main land, with entrances to suit the prevailing currents and winds," proposed forming "a hollow island in the ocean, which should enfold within its arms the ships of Britain;" for, he adds, "to attempt to form a harbour on any part of the coast, where there is an accumulation of beach, which is moved forward by the prevailing winds, * * * is useless."

When landings or promenade piers are built as trestle-work, on screw-piles or iron cylinders, and when parallel or converging jetties at river mouths or harbor entrances have their innermost portions submerged, the object may be to offer no obstruction to the passage of the moving material. Leaving aside such structures, however, and limiting consideration to works for the accommodation of shipping consisting of a solid outer portion in deep water and a relatively long, open inner portion, few are the cases where the principle has actually been applied in practice, but more numerous the projects which have appeared at different times and caused considerable discussion.

A pier built at Rosslare, in Ireland, according to Rendel's plans, as an open viaduct terminating in a solid breakwater with a depth alongside of 5 m. below low water, the total length being some 350 m., could, it seems, hardly be considered an entire success. Lately, the owners have been lengthening the pier and putting in a new viaduct.

A similar pier, built in 1887 at Ceara, in Brazil, some 930 m. in length, and extending originally to a depth of 5.8 m. at low water, is reported as having been entirely sanded up and abandoned.

Of projects for complete island harbors, may be mentioned J. Scott Russell's and Dupuy de Lôme's for Calais, A. Clarke's for Boulogne, Rymer Jones' and A. Clarke's for Madras, J. Coode's and J. Hawkshaw and Brunlee's for Port Elizabeth, in South Africa, all from the Seventies. In a competition for a prize, offered by the King of Belgium in 1881, three of the contestants, J. B. Redman, J. Scott Russell and A. Baldaque, recommended island harbors as the best solution of the problem, *i. e.*, how to improve harbors on low and sandy coasts.

Combinations of partly open and partly solid piers with detached breakwaters have been planned by Hélin for Ostende, and at various times by others for Dover and other places.

No such works, of any magnitude, however, seem to have been actually carried out. The writer has in vain searched the technical literature for records of any in existence, and is aware of only three little Danish fishing ports, which, on a diminutive scale, represent genuine complete island harbors. In his search, and on behalf of a committee appointed by the Danish Government, to investigate the matter, he addressed the Chief of Engineers of the United States Army in regard to American experience, and has to acknowledge the extraordinary courtesy shown him by Brig.-Gen. G. L. Gillespie, in sending letters of inquiry to a great number of officers of the Corps of Engineers. Exceedingly valuable as was the information collected about detached breakwaters, the investigation, as summed up by Capt. C. H. McKinstry, gave the expected, negative result, that island harbors and harbors formed by outlying breakwaters connected with the shore by open viaducts are without example in the United States.

In all the designs mentioned, the guiding principle has been to locate the solid works so far out as to be entirely outside the zone of littoral drift. The consonance of opinions seems to be, that it hardly leaves any doubt that this, if really accomplished, will prove effective to prevent silting up of the harbor, even though accumulations may form at the shore and the said zone may be widened locally. But, also, it leaves no doubt that the expense of such works will needs be considerable in proportion to the advantages gained. On account of the deep water at that distance out, the land area needed for the traffic, if the port is intended for commercial purposes and not for fishing boats or passenger transfer only, cannot be obtained without great cost by filling in; for, to form land, as suggested by Gaudard, by a temporary diversion of the sand movement, by a dam to be removed later, when the drift is to trend along the shore as originally, would probably meet with practical difficulties. Expensive, alike, are the high sea-walls, which from their nearly vertical faces reflect the waves and cause a choppy sea around the harbor. For these and other reasons, when, on sandy coasts, the choice has been between parallel jetties, converging jetties, detached breakwaters and an island harbor in deep water, the latter expedient has not been resorted to.

The question, however, arises naturally: Should island harbors

really be built necessarily outside the zone of littoral drift, or might they not lie within that zone and still not silt up? Experience and theory seem to prove that they may. The island harbors of the three little Danish fishing ports belong to this class, and, possibly, examples can also be found in other countries.

The Danish Isles and the Peninsula of Jutland have an area of only 38 460 sq. km. and a shore line of about 5 270 km. On a part, perhaps about one-seventh, of this length, particularly on the south coast of the Island of Bornholm in the Baltic, on the north coast of Zealand, facing the Cattegat, and on the north and west coasts of Jutland, facing the Skagerrack and the North Sea, construction of harbors is rendered difficult by the littoral drift. Here the island harbors are situated. Two of these, at Arnager and Snogebæk, on Bornholm, were built in a tentative way, in 1883 and 1888, whereas the third one, at Hundested, on Zealand, was formed in 1893 by the transformation of an originally land-connected harbor. All three are shown by Figs. 1, 2 and 3, and were built by Mr. H. Zahrtmann. The basins, enclosed by rip-rap moles, are from 1.3 to 2.5 m. deep, and cover areas of 0.11, 0.08 and 0.66 hectare, respectively; or, including the outer basin of the latter, 0.88 hectare. From the moles, open viaducts, wooden or composite, from 100 to 200 m. long, lead to the shore, and are divided into from 13 to 20 bays, which span openings from 6.3 to 9.4 m. wide.

On the coasts of Denmark, tides are insignificant, hardly perceptible in the Baltic, not exceeding ⅓ m. in the Cattegat and rising only to 1½ m. in the North Sea near the southern boundary. Hence, any movement of material, which takes place, is not due to tidal action but to the action of the waves, combined, perhaps, with that of local currents, and, whichever of these agencies be considered, attributable to the effect of wind.

The object aimed at by the three fishing ports seems to have been accomplished fairly well. At neither Arnager nor Snogebæk has material accumulated to an alarming degree; it is pure quartz sand, the size of the grains being 0.45 and 0.25 mm., respectively. At Hundested the drifting material is more heterogeneous, consisting of a mixture of quartz sand with grains of 0.25 mm. in diameter, gravel, shingle, larger pebbles and good-sized boulders. Some accumulation has taken place inside the 5-m. contour, and

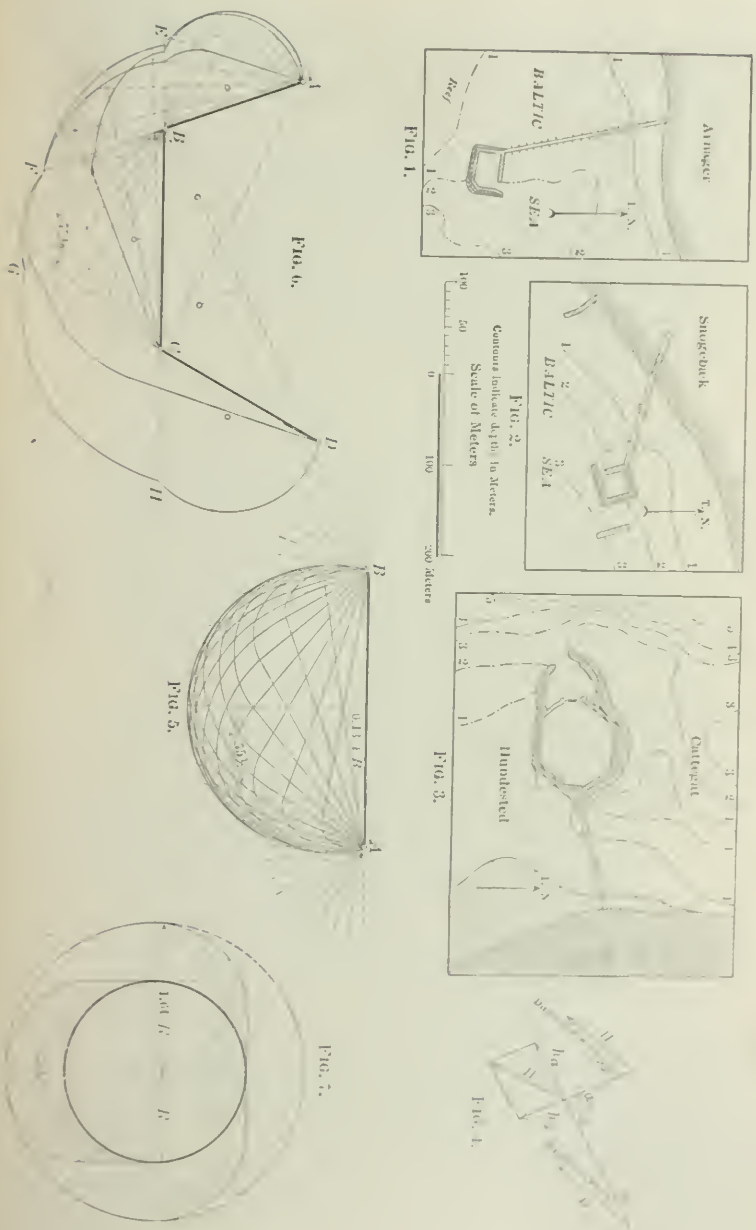

FIG. 1.

FIG. 2.

FIG. 3.

FIG. 4.

FIG. 5.

FIG. 6.

FIG. 7.

banks have formed at the southeast mole and at the shore, southeast of the port; but a state of equilibrium seems to have been reached, in which these two banks play an important part.

In this paper, the writer proposes to outline a method for estimating—without any claim of accuracy whatever—approximate limits for the accumulations which may form behind a detached, solid work, before a more or less stable equilibrium is established, provided the movement of material is due only to wave action. Such an estimate will be necessary for forming an opinion of how far out from the shore a given structure should be placed to insure against its becoming land-connected in time.

* * * * * * * * *

Littoral drift is due to currents and waves. The former, when caused by the wind, are in some measure directly-impelled drift or supply currents, or shore-currents within the line of breakers produced indirectly through the waves. Material transported by currents, either in suspension or rolling and sliding along the bottom, will form deposits or accumulations both in front and to leeward of any obstacle which deflects the stream lines.

Waves, as long as they remain oscillatory, do not transport material. Not until they have become translatory, when they have broken on a shelving beach at a depth of water perhaps equal to their height, are they capable of producing littoral drift. Approaching land, the translatory waves stir up material from the bottom and carry it forward in the direction in which they advance, or they push it by their impulse in the same direction, rolling and sliding along the bottom. The return wave, or the under current produced by the retreating water, aided by the slope of the beach, carries and drags backward the suspended and the heavier matter, respectively. Gravitation tends to make it follow the line of quickest descent, which is generally normal to the shore line. The material thus travels in a zigzag line, moving shoreward in the direction of the advancing waves, more or less inclined to the shore line, and retreating more nearly at right angles to the same than if reflected at an angle equal to that of incidence.

Accumulations due to wave action, when, though shifting always to a certain extent, they are possessed of some stability, are the

resultant effect of waves from different quarters and of different intensities striking the shore. Perhaps the product of the square of the velocity of the wind, which raised the waves, into the square root of the distance from the windward shore, according to Stevenson's law, may be taken, in the absence of any better, as an approximate measure of the quantity of material moved in a unit of time and on a unit of length. This measure adopted, what will be termed an "effect-rose" may be constructed, the radii of which are the products of the square roots of their respective fetches into the sums of the products of the squares of the different wind velocities into their relative frequencies. Suppose the resultant of the effect-rose found; its direction is that of a system of waves which, as far as littoral drift is concerned, to some extent replaces or is the resultant of all the systems actually striking the shore at different angles and during a certain length of time, say one year. Corresponding to that direction, the accumulations should preferably be determined.

Experience and theory prove that accumulations will form both at the outlying work and at the shore. Each of these accumulations, of which the latter is divided into parts, will be treated separately.

I.

When waves, translatory or oscillating, encounter an obstacle, the portions which are not intercepted, but pass onward, spread laterally under its lee and turn circularly around its extremities, thereby suffering a reduction of height. According to Stevenson, the height, h_a, in any direction, deflected α degrees from the original direction of advance of the waves, is approximately,

$$h_a = (1 - k\sqrt{\alpha})\, h_o \qquad (1)$$

where $k = 0.04$ to 0.06. The latter of these values is chosen in the sequel.

The mean velocity of the body of water below the crest of a translatory wave being approximately proportional to the height, h, when the depth is sensibly constant and not relatively too small, and the mean velocity of a water column at the intersection of two deflected waves being the resultant of the mean velocities corresponding to those two waves, the limits of the space, inside of which sediments of suspended material are apt to form, may be determined.

Suppose the water saturated with material, corresponding to the velocity of the waves striking the obstacle. It will deposit some of it as soon as the velocity decreases by deflection; and only where, at the intersections of the deflected waves, the resultant of their mean velocities equals or surpasses the mean velocity of the undeflected wave, will precipitation not take place. The crests of the two systems of waves deflected around the two extremities of the obstacle do not intersect in every point, but in a definite system of curves, dependent on the wave-length. As this latter may vary, however, it seems permissible to assume that every point becomes in time a point of intersection. Therefore, in determining the limiting curve for the sediments, an approximation may be had by taking the greatest mean velocities, *i. e.,* those below the crests. But instead of these, as stated above, the heights of the waves may be taken, as determined by Stevenson's formula.

The limiting curve, corresponding to any given direction of the incident waves, is determined by an equation between the angular co-ordinates, α and β, being the angles of deflection of the intersecting waves tangent to the obstacle. Of this curve, however, only the portion lying between the undeflected tangent waves, for which α or β vanish, should be considered. By reference to Fig. 4, the equation is deduced from the condition:

$$h_o^2 = h_\alpha^2 + h_\beta^2 + 2\, h_\alpha\, h_\beta \cos. (\alpha + \beta).$$

whence, by Stevenson's formula (Equation 1):

$$1 = (1 - 0.06 \sqrt{\alpha})^2 + (1 - 0.06 \sqrt{\beta})^2 + 2\,(1 - 0.06 \sqrt{\alpha}) \times (1 - 0.06 \sqrt{\beta}) \cos. (\alpha + \beta) \ldots\ldots (2)$$

Conjugate values of α and β are as follows:

α or β = ...	0°	10°	20°	30°	$37\frac{2}{3}$°	40°	50°	60°	70°	80°	90°
β or α = ...	$101\frac{1}{2}$°	73°	58°	46°	$37\frac{2}{3}$°	35°	26°	18°	$11\frac{1}{2}$°	6°	2°

If the obstacle be a vertical plane, *A B* (Fig. 5), the curves corresponding to different angles of incidence, all determined by Equation 2, are enveloped by a segment of a circle, subtended by *A B,* and containing an angle of $75\frac{1}{3}$ degrees. This arc, the center of which is 0.13 *A B* from the plane, is the limit of deposits for all possible directions of the waves.

Suppose the structure consisting of three vertical planes, *A B C D*

(Fig. 6). Suppose further, that Stevenson's formula holds good, however the waves have been deflected through the angle, α, *i. e.*, whether the deflection has taken place around one point or successively around several points. Then, for any given direction of the incident waves, Equation 2 has to be applied successively to A and B within the space, $A\,B\,E$, to A and C within $E\,B\,F$, to B and C within $F\,B\,C\,G$, to B and D within $G\,C\,H$ and to C and D within $H\,C\,D$. Hence, the limit of deposits for different directions of the waves is a system of five segments of circles each within one of the spaces, $A\,B\,E$, $E\,B\,F$, $F\,B\,C\,G$, $G\,C\,H$, $H\,C\,D$, subtended by $A\,B$, $A\,C$, $B\,C$, $B\,D$, $C\,D$ and containing angles of 75⅓ degrees.

Let the obstacle be a vertical circular wall (Fig. 7). Again, Equation 2 is to be applied, the directions of the deflected waves being all tangent to the circle. The curves corresponding to different directions of the waves are all identical, and their envelope is a circle concentric with the wall and containing between the two tangents from its points to the latter an angle of 75⅓ degrees.

Generally, whichever be the form of a vertical obstacle, the limit of deposits, corresponding to different directions of the incident waves, is a curve from the points of which the obstacle is seen in an angle of 75⅓ degrees.

If in Equation 1 the value, $k = 0.04$, be chosen instead of 0.06, the angle is 80° instead of 75⅓ degrees.

II.

Turning next to the accumulations forming at the shore, the coarser material moving along the bottom is to be considered.

For the present purpose the transporting capacity of a wave at a given point may be measured by the product of the weight of the material that it is able to move, into the velocity of the travel of that material perpendicularly to the coast and into that of the same parallel to the coast. The impulse of the wave being proportional to the square of its velocity, and the velocity being taken as proportional to the square root of its height, the transporting capacity may be expressed by:

$$E = h^2 \sin.\, \mu \cos.\, \mu \qquad (3)$$

where h is the height of the wave, and μ the angle of inclination of its direction of advance to the shore line. To each pair of values of E and h correspond two angles, μ and $90° - \mu$.

When the shore line has assumed a figure of equilibrium, the transporting capacity, E, is the same in every point of it. This condition determines the shape of the accumulation which is formed at the shore near an outlying work.

A wave, w, advancing toward an obstacle, $A\ B$ (Fig. 8), in a direction, $A\ D$, will, when A is passed, extend laterally beyond $A\ D$ as a circular arc with its center at A. If there were no obstacle, the height of the wave might remain unchanged during its advance from w_1 to w_2, the part of it, $C\ F$, being annihilated by the beach. But when it extends laterally behind an obstacle, some of the energy contained in $A\ C$ is consumed, its height decreases and its effect on the beach is reduced. Therefore an accumulation will form from C toward D, the shore line advancing seaward, until the gap between it and the structure has been narrowed, and the height of the wave thereby increased, sufficiently to make the transporting capacity, E, in any point, corresponding to the inclination of the shore line there, the same as it was before the outlying work was built.

As at A, the waves at B will extend inside $B\ E$ as circular arcs with the center at B. This, however, does not sensibly diminish their height outside $B\ E$, inasmuch as they extend practically infinitely that way. These circular extensions of the wave crests at A and B are nothing but the deflected waves, dealt with previously in connection with the suspended material. Again, their interference, where they intersect, is to be considered, and Stevenson's formula, Equation 1, applies. But the condition imposed here being different from that expressed by Equation 2, another curve is determined by it. Within the field, $A\ B\ E\ D$, the figure of equilibrium of the shore line is such that in any point of it the transporting capacity of the interfering waves is the same as outside the sphere of influence of the work.

Where this shore line cannot be reached directly by waves deflected at A or B, because it is sheltered by itself or by the structure, the littoral drift is due to the interference of waves successively deflected at two or more points with others deflected at one or also successively at more points.

Waves deflected at B do, of course, pass beyond $A\ D$. But, advancing, as they may be supposed to do, partly against the wind, they vanish quickly when not sheltered any more by the structure,

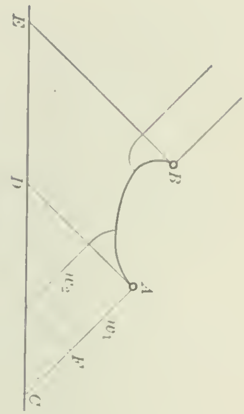

Fig. 8

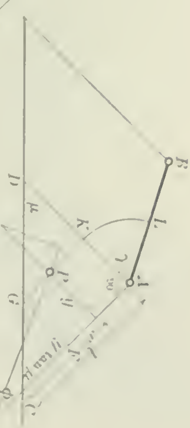

Fig. 9

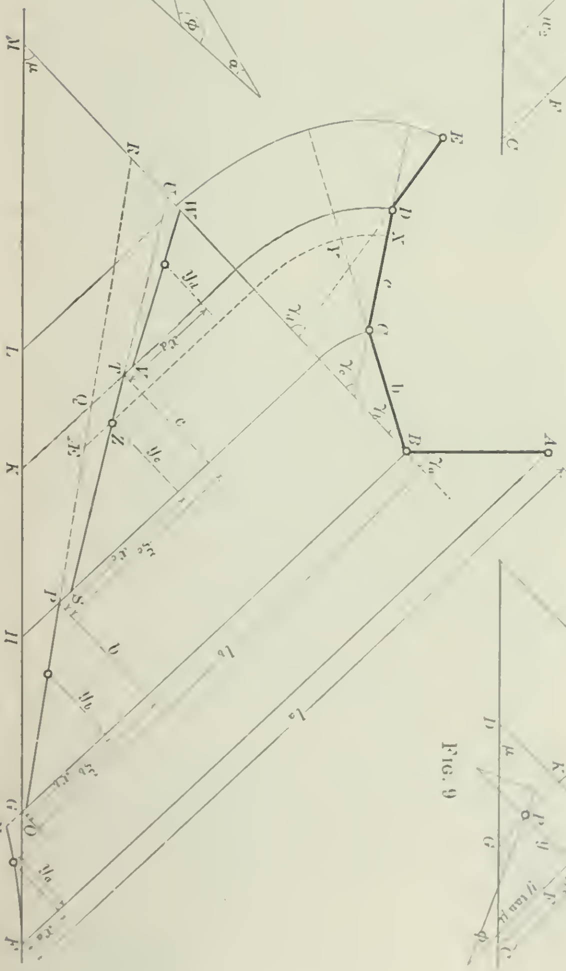

Fig. 10

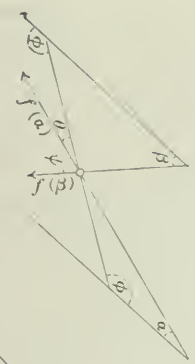

Fig. 11

and therefore need not be considered within the field, *A C D*. Similarly, waves deflected at *A* pass beyond *B E;* as they advance more nearly with the wind, they may perhaps exert some influence beyond and near *E,* where the angle of deflection is relatively small.

The accumulations within the fields, *A C D* and *A B E D,* are now to be considered separately.

The Field, A C D.—Stevenson's renowned formula for the reductive power of harbors, according to which, when its second member is left out of consideration, the reduced heights of the waves inside the harbor entrance are approximately in the inverse ratio of the square roots of the breadths of the harbor at the respective places, hardly applies in the present case, where the question is not of ordinary, oscillatory waves, but of translatory rollers. It seems more appropriate to assume the height to be in the inverse ratio of the breadth, or, when the height is different in different parts of the wave, to assume the total volume of water moved in the whole breadth constant.

Let the obstacle be a vertical plane, *A B* (Fig. 9), and the direction of advance of the waves form angles, γ and μ, with the plane and with the original shore line, respectively. Again, let the shore line have advanced to a point, *P*, the co-ordinates of which are x and y, and let h_y be the height of the wave, *L K P*, which, if it had not extended through *K L* and at the same time been reduced by *G P*, should have had a height, *H*, and a breadth, *G K*. Then h_y is determined by:

$$\left[\int_o^{\gamma} (1 - 0.06 \sqrt{\alpha})\; y\; \frac{d\alpha}{57.3} + l - x\right] h_y = (l - y \tan.\, \mu)\; H,$$

whence, by integration:

$$h_y = \frac{l - y \tan.\, \mu}{\dfrac{\gamma - 0.04\; \gamma^{\frac{3}{2}}}{57.3} y + l - x}\; H \ldots\ldots\ldots\ldots\ldots (4)$$

If the shore line at *P*, when protracted, forms an angle, φ, with *A C*, and $90° - \varphi$ with the direction of advance of the waves, the condition of equilibrium, *i. e.*, that the transporting capacity at *P* is the same as beyond *C*, is according to Equation 3:

$$h_y^2 \cos.\, \varphi \sin.\, \varphi = H^2 \sin.\, \mu \cos.\, \mu \ldots\ldots\ldots\ldots\ldots (4a)$$

or, by Equation 4:

$$\frac{l - y \tan.\, \mu}{\dfrac{\gamma - 0.04\; \gamma^{\frac{3}{2}}}{57.3} y + l - x} = \sqrt{\frac{\sin.\, 2\, \mu}{\sin.\, 2\, \varphi}} \ldots\ldots (5)$$

This condition, by the substitution,

$$\sin. 2\,\varphi = \frac{2\dfrac{dy}{dx}}{1 + \left(\dfrac{dy}{dx}\right)^2}$$

gives the differential equation of the shore line of equilibrium. It involves only the first differential, but is quadratic and hardly admits of integration. An approximate solution, however, may be found by constructing a series of tangents, the inclinations of which vary by degrees.

For the system of curves represented by the differential equation, the locus of points such that at each of them the tangent to the curve through it has a direction corresponding to a given value of φ, is a right line determined by Equation 5 with φ constant, or say:

$$x = \left(\frac{\gamma - 0.04\,\gamma^{\frac{3}{2}}}{57.3} + \tan.\,\mu \sqrt{\frac{\sin.\,2\,\varphi}{\sin.\,2\,\mu}}\right) y$$
$$+ \left(1 - \sqrt{\frac{\sin.\,2\,\varphi}{\sin.\,2\,\mu}}\right) l \ldots\ldots\ldots\ldots\ldots\ldots(6)$$

It intersects $A\,C$ and $A\,D$ at distances, $l - x$ and y, from A, with $y = 0$ and $x = l$, respectively. Such lines, corresponding to different values of φ, being drawn, the shore line may be found graphically, by starting out tentatively from points near C. Inasmuch as the intersections of the lines and $A\,C$ are nearer C, the larger $\sin.\,2\,\varphi$ is, the right line corresponding to $\sin.\,2\,\varphi = 1$ will differ less from the original shore line than any one of the others and than any of the curves determined by the differential equation. The figure of equilibrium, first reached by the advancing coast and remaining the final one, is of all the curves, of which no two of the same system intersect, the one which intersects $A\,C$ as near as possible to C and $A\,D$ as near as possible to its intersection with the right line corresponding to $\varphi =$ 45 degrees.

One of the curves, however, is itself a right line, the equation of which is Equation 6, with the angle, φ, determined by equating the coefficient of y to $\cot.\,\varphi$. Thus:

$$\cot.\,\varphi - \frac{\tan.\,\mu}{\sqrt{\sin.\,2\,\mu}} \sqrt{\sin.\,2\,\varphi} = \frac{\gamma - 0.04\,\gamma^{\frac{3}{2}}}{57.3} \ldots\ldots\ldots(7)$$

which is of the fourth degree in $\tan.\,\varphi$. This right line may presumably in many cases be taken as a sufficient approximation to the least advanced line of equilibrium, *i. e.*, to the actual shore line.

If the obstacle consist of four vertical planes, $A\ B\ C\ D\ E$ (Fig. 10), the waves in advancing from $A\ F$ to $B\ G$ swell and erode the beach until its curve is such that at any point of it (x, y), the wave height, h_y, and the inclination, φ, of the tangent to the crest, $A\ F$, are given by:

$$h_y = \frac{l_a - y_a \tan.\ \mu}{l_a - y_a \tan.\ \gamma_a - x_a} H;$$

and, by Equation 3,

$$\frac{l_a - y_a \tan.\ \mu}{l_a - y_a \tan.\ \gamma_a - x_a} = \sqrt{\frac{\sin.\ 2\ \mu}{\sin.\ 2\ \varphi}} \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

Equation 8 is solved in a way similar to that used for Equation 5. For the present purpose, it may be preferable, generally, not to take this erosion into account. But if it is considered, in the subsequent determination of the new shore line within the triangle, $B\ G\ M$, l_b should be replaced by $B\ N$, and μ by the angle between the tangent to the eroded shore line at N and the direction of advance of the waves.

Leaving out the erosion, the advanced shore line is first determined within the quadrilateral, $B\ G\ H\ C$, by Equations 5, 6 and 7, when l_b and γ_b are substituted for l and γ. Let $O\ P\ R$ be the right line thus found as an approximation to the shore line, of which, however, only a part, $O\ P$, to the intersection with $H\ C$, is actually formed.

Within $C\ P\ Q\ D$, the right line, $S\ T\ U$, is next found by the equation:

$$\frac{l_b - (y_c + b) \tan.\ \mu}{F(\gamma_c)\ y_c + F(\gamma_b)\ b + l_b - \xi_b - x_c} = \sqrt{\frac{\sin.\ 2\mu}{\sin.\ 2\varphi}} \ldots\ldots (9)$$

where, for brevity's sake, $F(\gamma)$ stands for $\dfrac{\gamma - 0.04\ \gamma^{\frac{3}{2}}}{57.3}$. It expresses the fact that the quantity of water in a wave-crest, $Y\ Z\ Æ$, given by the equations for $B\ G\ H\ C$, is the same as that in the crest, $X\ Y\ Z$, of the altered wave, when it is remembered that $X\ Y$ is a circular arc with its center at C, in which the wave is deflected through an angle varying from γ_b to γ_c. Due to their analogy, Equations 9 and 5 are solved similarly, and Equations 6 and 7 may be applied, when properly modified.

Within $D\ T\ U\ E$, the right line, $V\ W$, is determined in the same way by the equation:

$$\frac{l_b - (y_a + c + b) \tan. \mu}{F(y_a)\, y_a + F(y_c)\, c + F(y_b)\, b + l_b - \xi_b - \xi_c - x_a} = \sqrt{\frac{\sin. 2\mu}{\sin. 2\varphi}} \quad (9a)$$

and so on for every angle in the figure of the work.

If the obstacle be a vertical circular wall, the equation is:

$$\frac{l - y \tan. \mu}{\frac{y^2}{r}\left(0.5 - 0.12 \sqrt{\frac{y}{r}}\right) + l - x} = \sqrt{\frac{\sin. 2\mu}{\sin. 2\varphi}} \quad \ldots\ldots\ldots (10)$$

Here l is the length, between the shore line and the wall, of a wave-crest through the center of the circle; r is the radius of the latter. The abscissas are measured along that wave-crest from its intersection with the shore line, and the ordinates are perpendicular to it.

The Field, A B E D.—As the waves deflected around A and B (Fig. 8) advance, maintaining the same distance between consecutive crests, the points of intersection of the crests describe a system of curves. The shore line, where intersected by these curves, is struck simultaneously by two wave-crests, and the material is moved by the resultant of the two wave motions. But between such points of the shore line are others, struck at different times by crests of the two wave systems and where different phases of the wave motions are therefore compounded to a resultant.

At a point both waves may, in the extreme cases, be in the same phase or in opposite phases, either of advance or return; hence four different cases arise. According to the law of zigzag travel of material along the beach, the return movement is approximately normal to the shore line and not opposite that of advance. Keeping this in mind, the distance may be deduced, which a particle travels in each of the four cases during one period, *i. e.*, the time intervening between two wave-crests intersecting at a point and the two next ones intersecting at the same point. If $f(\alpha)$ and $f(\beta)$ signify the effect of the waves coming from A and B, respectively, ϑ and ψ their angles of incidence with the shore line, the distance traveled in a direction parallel to the shore is:

$$f(\alpha) \cos. \vartheta + f(\beta) \cos. \psi,$$

in all four cases, and in a direction normal to the shore:

$$\pm f(\alpha) \sin. \vartheta \pm f(\beta) \sin. \psi,$$

the same signs applying to two of the cases and the opposite signs to the other two. The mean travel of material in one period, for all the

points of the shore, may hence be taken as approximately the same as above, parallel to the shore, and:

$$\tfrac{1}{2}\,[f(\alpha)\sin.\theta + f(\beta)\sin.\psi \pm f(\alpha)\sin.\theta \mp f(\beta)\sin.\psi] = \begin{cases} f(\alpha)\sin.\theta \\ f(\beta)\sin.\psi \end{cases}$$

in a direction normal to the shore, the greater of the two values being always used in the latter expression.

The transporting capacity is the product:

$$[f(\alpha)\cos.\theta + f(\beta)\cos.\psi] \times \begin{cases} f(\alpha)\sin.\theta \\ f(\beta)\sin.\psi \end{cases}$$

corresponding to $f(\alpha)\sin.\theta \gtrless f(\beta)\sin.\psi$, respectively.

In this expression, the angles, θ and ψ, may be replaced by the angles of deflection, α and β, and the angle, φ, between the direction of advance of the undeflected waves and the shore line at the point considered, inasmuch as (Fig. 11):

$$\theta = 180^\circ - \varphi - \alpha; \text{ and } \psi = 180^\circ - \varphi + \beta.$$

Equating finally the transporting capacity inside to that outside the sphere of influence of the work, the following equations are deduced:

$$[f(\alpha)\cos.(\varphi+\alpha) + f(\beta)\cos.(\varphi-\beta)] \times \begin{cases} f(\alpha)\sin.(\varphi+\alpha) \\ f(\beta)\sin.(\varphi-\beta) \end{cases} = -H^2\sin.\mu\cos.\mu \;\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

corresponding to, respectively:

$$f(\alpha)\sin.(\varphi+\alpha) \gtrless f(\beta)\sin.(\varphi-\beta) \ldots\ldots\ldots (12)$$

By transformation, Equation 11 is given the more convenient form:

$$a\sin.2\varphi + b\cos.2\varphi = c \;\ldots\ldots\ldots\ldots\ldots (13)$$

where:

$$a = \begin{cases} f^2(\alpha)\cos.2\alpha + f(\alpha)f(\beta)\cos.(\alpha-\beta) \\ f^2(\beta)\cos.2\beta + f(\beta)f(\alpha)\cos.(\beta-\alpha) \end{cases}$$

$$b = \begin{cases} f^2(\alpha)\sin.2\alpha + f(\alpha)f(\beta)\sin.(\alpha-\beta) \\ -f^2(\beta)\sin.2\beta - f(\alpha)f(\beta)\sin.(\beta-\alpha) \end{cases}$$

$$c = \begin{cases} -H^2\sin.2\mu - f(\alpha)f(\beta)\sin.(\alpha+\beta) \\ -H^2\sin.2\mu + f(\beta)f(\alpha)\sin.(\beta+\alpha) \end{cases}$$

all the upper and all the lower of these expressions corresponding.

The solution of Equation 13 being:

$$\sin.2\varphi = \frac{ac \pm b\sqrt{a^2+b^2-c^2}}{a^2+b^2} \ldots\ldots\ldots\ldots (13a)$$

each of the two equations gives four values of φ, two of which, however, are extraneous. Of the remaining two pairs of values, some do not satisfy the conditions of the Inequality 12, and hence are inapplicable. The one or two roots still left determine, at the point

considered, the directions of tangents to possible shore lines of equilibrium.

The functions $f(\alpha)$ and $f(\beta)$ are taken to be the heights of the deflected waves. Stevenson's formula, Equation 1, applies, if the waves are deflected directly from $A\ D$ and $B\ E$ (Fig. 8), h_{y} being the height of their undeflected portion, while H is the height outside the influence of the work. Hence:

$$f(\alpha) = (1 - 0.06\sqrt{\alpha})\, h_y; \text{ and } f(\beta) = (1 - 0.06\sqrt{\beta})\, H;$$

where h_y is the same as determined for the field, $A\ C\ D$, by Equations 8, 4, 9, 9*a*, etc., or more simply by Equation 4*a*. If the waves are not deflected directly, say from $A\ D$, but successively around different points through the angles, $\alpha_0, \alpha_1, \alpha_2, \ldots$ so that $\alpha = \alpha_0 \pm \alpha_1 \pm \alpha_2 \pm \ldots$ the function, mostly on account of the opposite directions of the partial deflections, is taken as:

$$f(\alpha) = (1 - 0.06\sqrt{\alpha_0})\,(1 - 0.06\sqrt{\alpha_1})\,(1 - 0.06\sqrt{\alpha_2}) \ldots h_y.$$

Under similar circumstances, an analogous expression may be had for $f(\beta)$.

Now, within the field, $A\ B\ E\ D$, the curve of the shore line, when a polygon formed of sufficiently short parts of its tangents is substituted for it, may be determined approximately by the somewhat slow procedure of calculating φ successively from Equation 13, after having measured α and β each time on a drawing, on which the tangents are being laid down *pari passu*. It starts out from the point of intersection of $A\ D$ and the shore line within the field, $A\ C\ D$, as determined under the heading of that field.

If, at a point of the curve, Equation 13*a* gives only one applicable value of φ, there is through that point only one curve of more or less unstable equilibrium. But, if Equation 13*a* gives two values of φ, two curves of equilibrium are possible. Of these, the one representing an unstable equilibrium may be the form of the shore line as first established; but the tendency will be toward approaching the other one, which represents a stable equilibrium, that is where an erosion decreases and an accumulation increases the transporting capacity.

It is to be remarked that, in this investigation of the shore line of equilibrium, attention has only been paid to the drift along the shore, leaving the suspended material entirely out of consideration. Where there is nothing but coarse material, or the fine ma-

terial is so fine that it does not form deposits at the obstacle, the assumption is warranted. But, where, on the lee side of a work, a deposit is formed within the limiting curve from the points of which the work is seen in an angle of $75\frac{1}{3}°$, such deposit will modify the form of the shore line within *A C D,* and, consequently, within *A B E D* as well. The advance of the shore will be somewhat less than calculated. However, as the error is thus on the safe side, and as, at times, the waves may, perhaps, be so high as to carry away the deposits of suspended matter, the shore line determined may be taken as the approximation required.

Examples.

Some examples of the methods just described for calculating and forming the accumulations caused by outlying works are shown in Figs. 12, 13 and 14.

In the first case the obstacle is a vertical plane, *A B* (Fig. 12). $\mu = 40°$, $\gamma = 58$ degrees. For the right line within the field, *A C D*, Equation 7 gives $\varphi = 33\frac{1}{2}°$, and Equation 6, the distances from *A*, of its intersections with *A C* and *A D*, $0.97\,l$ and $0.64\,l$, respectively. Within the field, *A B E D*, Equation 13 gives successively:

	F G						*G H*							*G K*					
α	0	2	4	6	8	10	11	12	14	17	19	22	24	11	0 = 11–11	4 = 11–7	7 = 11–4	7 = 11–4	$7\frac{1}{2}$ = 11–$3\frac{1}{2}$
β	65	60	54	48	43	38	35	33	28	22	17	11	6	35	32	28	24	21	18
φ	$155\frac{1}{2}$	155	151	146	141	137	$134\frac{1}{2}$	132	$127\frac{1}{2}$	$121\frac{1}{2}$	$116\frac{1}{2}$	110	105	180	173	172	172	171	170
φ^1	....	....	....	....	....	....	180	179	176	172	169	166	$163\frac{1}{2}$	$124\frac{1}{2}$	145	137	130	127	$123\frac{1}{2}$

From the point, *G*, determined by the co-ordinates $\alpha = 11°$, $\beta = 35°$, two limit-curves depart, one of which, *G H*, represents a possible, the other, *G K*, a stable equilibrium. Points of the latter are struck by waves deflected not directly around *A*, but first 11° around *A* and then from 11° to $3\frac{1}{2}°$ in the opposite direction around *G*. In every point of *G H* and *G K*, φ has two values one corresponding to unstable, the other to stable equilibrium.

The second case (Fig. 13) represents the actual conditions of the island harbor at Hundested. The rhumb of the resultant of the effect-rose is from N. 60° W. Its angle of inclination to the original shore line is measured as 54° or 39°, according as one or another portion of the latter is considered; but as the transporting capacity (Equation 3) must be the same for both portions, one angle should be the complement of the other; thus the values are cor-

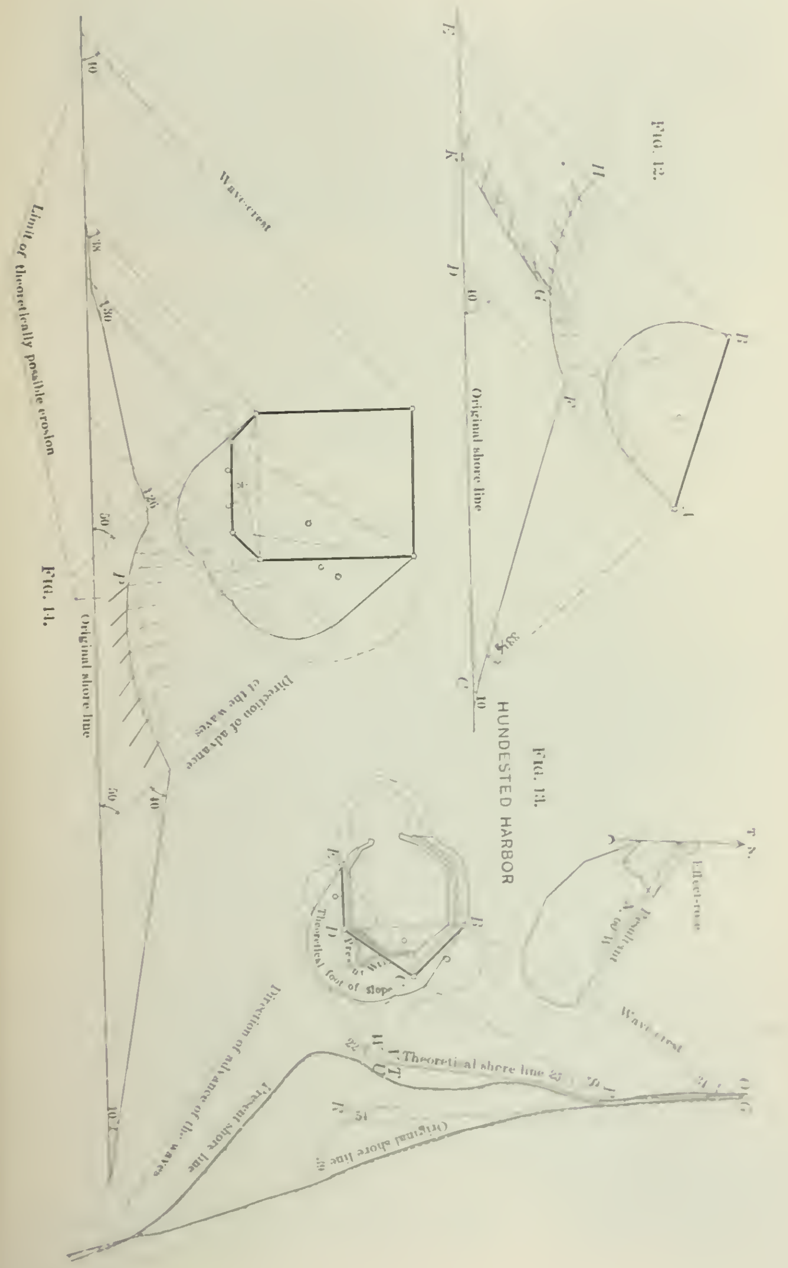

FIG. 11.

FIG. 12.

FIG. 13.

rected a trifle. Hence $\mu = 53$ degrees. By Equations 5, 6, 7, 9 and 9*a*, the right lines within the different quadrilaterals, formed by the sides of the work and the waves through its vertices, are determined, their angles with the wave-crests being $\varphi = 34^\circ, 25^\circ$ and 22 degrees. Connecting the lines by transition curves completes the construction of the first part of the theoretical shore line. The close agreement of the latter and the actually formed accumulation, as well as of the theoretical limit and the actually formed deposit at the work, seems gratifying.

The third case (Fig. 14) represents a planned harbor, where $\mu = 50$ degrees. Leaving out of consideration a great erosion which might take place, the most advanced theoretical shore line of unstable equilibrium is constructed. From the point, *P*, departs the line of stable equilibrium, approaching the original shore line.

* * * * * * * * *

To sum up: A detached work may cause certain deposits and accumulations to form, the limits of which, by the methods indicated, may be determined theoretically with some approximation, provided wave-motion is the transporting agency. Lest the work be apt to become land-connected in time, these limits of the deposits and accumulations must not overlap. More than this; there shall be a certain minimum distance between them. For it should be remembered, that, while the curve of the deposit represents the foot of the slope, the curve of the accumulation, forming a continuation of the original shore line outside the influence of the work, is the contour of the mean water level. Hence the distance between the two curves should be at least equal to the length of the natural slope of the beach out to what may in each case be considered a proper depth of water, that is the limit of the zone subject to littoral drift of coarse material.

With due regard to the littoral current, running approximately parallel to the original shore line, and to the deposits or accumulations formed by material carried along by it in front and to leeward of the deposits and accumulations caused by the waves, it may be even safer to demand, that the projections of the two curves, on a line at right angles to the general trend of the original shore line, must not overlap.

This enables one to judge whether a proposed detached work, such as an island harbor, is planned far enough out from the shore.

TRANSACTIONS

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HARBORS.

HARBOURS OF GREAT BRITAIN.

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When asked to prepare a paper on "Harbours," for the International Engineering Congress, the writer understood that what was required of him was, a reference to the present practice with regard to harbour construction as generally adopted by English engineers.

Obviously, it would be quite impracticable, within the limits of a paper of reasonable length, to enter upon a consideration, even in general terms, of the principal of the existing harbours around the seaboard of Great Britain and her colonial possessions. It is proposed, therefore, to confine this paper to some of the leading features which affect harbour construction generally, and to refer, by way of illustration, to a few works, either in course of execution, or which have been recently completed, with the details of which the writer is personally acquainted.

General Remarks on Harbour Design.

Although it may be practicable to indicate a few general conditions which affect the design of a harbour at any particular site, it

should be clearly understood that a thorough investigation of the physical conditions, in each case, is absolutely necessary before a suitable design can be prepared. This examination should have especial reference to the exposure of the site and the wave force which has to be provided for, the action of the tides, the set and velocities of the currents, the character of the sea bed, the possibility of shoaling consequent upon the proximity of sand or shingle accumulations, the nature of the shelter required and its extent, the character of the materials and of the local labour available, and many other points, each of which demands careful and minute investigation.

The extent of the accommodation, in order to provide for future growth in the size of shipping, should likewise be kept prominently in view, for, although, in the past, there has been an enormous development in this respect, further extensions should certainly be looked for in the future, especially as the growth in the size and tonnage of shipping will not improbably be governed and limited by the extent of harbour and dock accommodation which is available.

In this last-named connection, it may be observed that the enormous increase in the size and tonnage of steamers within the last few years, and latterly, also in a marked degree, in their draft, has necessitated, in many cases, considerable alterations and additions to existing harbours, docks and locks, so that in framing a design for harbour accommodation in these days, it is most important to keep in view the not improbable contingency of future extensions, which may be so engrafted on the first section of the works, as to harmonize therewith, when the extended requirements of trade are such as to justify the expenditure required in this further development.

Considerable variation in the heights of waves, the forces which they exert and the depths to which their action extends, is, of course, naturally to be met with along a seaboard where the exposure varies so greatly as is the case around the shores of Great Britain.

Taking two or three cases in point, it may be remarked that at Dover, a sea of greater height than 15 ft. from crest to trough has not been recorded since the new harbour works were commenced in 1893; whilst at the mouth of the Tyne, in connection with the North Pier works, waves of from 35 to 40 ft. from crest to trough have

been observed; at Peterhead, also, where a harbour of refuge is being constructed by the Admiralty, waves have been recorded which closely approximate to 40 ft. in height from crest to trough.

The late Mr. Thomas Stevenson suggested a formula by which the height of waves might be calculated, when the length of the fetch is known, by taking that height, in feet, from crest to trough, as one and a half times the square root of the fetch in nautical miles, where the conditions are favourable with regard to depth for wave development, and the time during which the storm lasts is sufficient for the production of waves of maximum force.

Mr. Stevenson's formula, which is generally used to calculate the extent to which waves will be reduced within a harbour, after passing through its entrance, is as follows:

$$x = H\frac{\sqrt{b}}{\sqrt{B}} - \frac{\left(H + H\frac{\sqrt{b}}{\sqrt{B}}\right)\sqrt[4]{D}}{50}$$

Where H = height of wave at entrance, in feet.

b = breadth of entrance, in feet.

B = breadth of harbour at place of observation, in feet.

D = distance from mouth of harbour to place of observation, in feet.

x = reduced height of wave at place of observation, in feet.

The result obtained should be considered, however, in connection with the wind wave generated within the sheltered area itself, which sometimes would be in the same direction, or practically so, as the run of the waves entering the harbour, and might materially influence the result, especially in the case of large harbours where the disturbance caused by these wind waves alone, irrespective of that due to seas passing through the entrance, becomes an important factor for consideration, especially when shelter is required along the leeward margin of the enclosure.

In connection with the design of harbours, it is, of course, important that spending beaches and slopes should be provided, where necessary, for dissipating "scend" or undulation within the enclosure, and that wave traps should be formed in the vicinity of internal entrances to docks or basins, in connection with such enclosures, where required.

There can be no question as to the depths to which wave action

extends, being much greater than was formerly believed to be the case. The late Sir James Douglass, who was a keen observer, and who, as Engineer-in-Chief to the Trinity House, had special opportunities for noting the effect of heavy seas in exposed positions, once referred, in the Institution of Civil Engineers, to lobster pots, off the Lands End, lying in from 20 to 30 fathoms, having been found to be filled with sand and shingle on their withdrawal subsequent to a heavy gale, some of the stones weighing as much as 1 lb., thus showing in that position, that sea action had extended to the depth named. Subsequent investigation on the spot by the writer confirmed this statement.

Sir James, at the same meeting, also gave a remarkable instance of coarse sand having been found on the external gallery of the Bishop Rock Light-House, off Scilly, after a gale, at a height of 120 ft., the water in the vicinity of the rock being 25 fathoms, thereby showing that the sea bed had been disturbed at that depth, this being the only source from which the sand referred to could have been obtained.

The most vulnerable portion of a sea work, during construction, is the "scar," or unfinished end. At the Peterhead Breakwater, during a storm which occurred in 1898, blocks weighing upwards of 41 tons were displaced at a level of 36 ft. 7 in. below low water of spring tides, the rise of such tides being 11 ft., and a section of the breakwater weighing 3 300 tons was slued bodily, 2 in. in extent, without the blockwork being dislocated. Calculations showed that, to effect such movement, a wave force of fully 2 tons per sq. ft. must have been exerted over the area so affected.

At Colombo, during the carrying out of the southwest breakwater, a length of wall 24 ft. in width, founded at 20 ft. below low water, the rise of tide being 2 ft., was canted inwards by sea action to an extent of 15 in. at the outer end, the portion of the work affected being 150 ft. in length, landward of which no movement occurred. The blocks were subsequently lifted and reset on a true line.

The foregoing instances have been mentioned, out of many others which might be cited, to show the enormous disturbing forces to which sea works are exposed in certain positions, and the great depths to which the action of waves extends where the conditions are favourable to their development.

With reference to the movement, or travel, of beaches, along the coasts of England, two opinions are held by engineers, one being that the travel is caused by wave action, the other attributing it to current action.

Along the south coast of England the movement is from west to east, which corresponds with the direction of the flood tide. On the east coast the movement of the beaches, except in the case of special indentations, is from north to south, which is also in the direction of the flood tide. The heaviest seas experienced on the south coast come from the southwest, and on the east coast from the northeast, and, therefore, it follows that the direction of the heaviest waves and of the flood tide happen, both on the south coast and on the east coast, to be in the same direction.

Extensive observation has shown, however, that the propelling influence which moves beaches along these shores is, undoubtedly, the oblique impingement of the prevailing and predominating waves, *viz.*, from the southwest, causing an easterly travel on the south coast, and from the northeast, causing a southerly travel on the east coast.

Instances could be cited in the writer's experience, where, on cutting off the propelling influence by the provision of shelter from the quarter of the prevailing winds, waves generated by strong winds in the opposite direction then become the predominating influence, and cause a reverse travel of the beach. This reverse travel has also been frequently observed, on open coast lines, when the wind has happened to blow for a lengthened period in a direction contrary to the prevailing wind, thus acting for a time against the normal travel of the beach.

Moreover, if the movement of beach along a shore had been due to current action, as distinguished from wave force, then the travel should be in operation in smooth water, whereas, under such conditions, practically no movement occurs.

With regard to the absence of shoaling on the leeward side of a work, this result is, of course, determined by the quantity of shingle which is travelling, and the exhaustibility, or otherwise, of the source of supply.

On the south coast of England a large quantity of beach is now being lodged on Dungeness Point, which, for a considerable period, has been accreting. From thence, generally, to Folkestone, costly

groynes and sea works have been necessary to protect the foreshore, the groynes being required for the trapping and maintenance of the shingle, which affords the natural protection to the sea front.

At Folkestone there has been a considerable growth of beach in years past, but at the present time, due to the trapping of the beach on the windward side, for the protection of the length of foreshore and sea front to which reference has been made, the extent of the travel is but little more than equal to what is required for the supply of material for repairs and formation of road surfaces. At present the accumulation to the westward of Folkestone Pier is not increasing to any appreciable extent, and is, therefore, not likely to prove serious in forming an obstruction at the new pier, or at the harbour entrance, an important consideration, especially in view of the extensive works which have been recently completed at this port, for the improvement of the cross-channel service.

Westward of the Admiralty Pier at Dover, the growth which is due to the waste of the foreshore between it and Folkestone is insignificant, and need not give rise to the slightest anxiety with reference to future accumulations proving prejudicial to the harbour there. At the present time a portion of the foreshore between Folkestone and Dover has to be groyned for its preservation.

Instances could be given of harbours along the New Zealand Coast, where considerable trouble has arisen from the growth of shingle on the windward sides of structures, which, in order to prevent the blocking of berthage on the inner face, have necessitated considerable outlay on extensions. In these cases, however, the supplies of beach material have been practically inexhaustible.

In the preparation of designs for pier or breakwater works, it is important, therefore, to consider:

a.—The direction of travel of the adjacent shingle arising from sea action from the most exposed quarter; and

b.—The extent of such travel as affecting, or not affecting, the future works.

Movement of sand is sometimes caused by current action, leading to a deposit in closed harbours, or in the vicinity of breakwaters or piers, which action is accelerated by waves disturbing the sand, the latter being carried in suspension until the necessary quiescence is produced to cause settlement. This is another, and sometimes

subtle influence, which requires most careful consideration by the engineer who is about to design sea works.

In laying out the lines of harbour structures, and determining the relative positions and lengths of the works which are required to afford the necessary shelter and accommodation, it is of great importance to keep fully in view, the position, width and aspect of the entrance, or entrances, to the proposed harbour. In certain cases it is practicable, especially where the proposed enclosure is of large extent, so to arrange the works as to admit of the adjustment of the widths of the entrances as experience may show to be desirable during construction, by extending one or other of the two converging moles or piers.

Bar Harbours and Pump Dredging.

Since pump dredging was initiated in Holland, about 1880, considerable advances have been made in perfecting dredge craft specially adapted for this class of work. Bar harbours, which before the introduction of this system of dredging were only partially successful, with reference to the works executed at their entrances for scour and training purposes, have, in consequence of the aid afforded by pump dredging, now become thoroughly satisfactory in the effects produced.

Two instances may be cited in illustration of this statement, *viz.,* the entrance to the Port of Durban on the Natal Coast, and the entrance to the Port of East London on the South African Coast. Probably no bar entrances existed of a less favourable character than at these two ports before their successful treatment was initiated, first, by the construction of moles and training works, and, secondly, by pump dredging.

To show the development which has taken place in the dimensions, capacity and power, of the dredging plant at East London, a list of the pump-hopper dredgers which have been used there is given in Table 1.

It will be seen from this table, that whereas in 1886 the first pump dredger was of 450 tons hopper capacity, the latest dredger, which was supplied for this port in 1903, is of 2 000 tons capacity. In this last-named connection it may be remarked that, in dredging on bars in exposed positions, the size of the dredge craft exercises

a most important influence, seeing that a large vessel can be worked in sea disturbance where it would be quite impracticable to use a smaller craft. For this reason, and also from motives of economy in working, the development to which reference has been made, has been brought about at East London.

TABLE 1.

Name of dredger	*Lucy.*	*Sir Gordon.*	*Kate.*	*Agnes.*
Year when built	1886	1890.	1897.	1903.
Length, in feet, over all	148	163	204	287
Breadth, in feet	30	32	39	44
Depth, in feet	$11\frac{1}{4}$	$11\frac{1}{2}$	$14\frac{1}{2}$	$18\frac{3}{4}$
Loaded draft, in feet	$10\frac{1}{12}$	10	$12\frac{1}{4}$	$15\frac{5}{12}$
Number of suction pipes	1	1	2	1
Diameter of suction pipes, in inches	20	24	27	48
Position of suction pipe	Side.	Side.	Side.	Center.
Depth, in feet, to which machine will dredge	30	35	40	40
Hopper capacity, in tons	450	500	900	2 000
Speed, in knots	7	$7\frac{1}{2}$	10	10
Propellers	Single.	Single.	Twin.	Twin.
Engines	One set, compound.	One set, compound.	Two sets, triple-expansion	Two sets, triple-expansion.
Indicated horse-power, propelling	400	420	1 150	1 850
Number of boilers	2	2	2	2
Diameter of boilers, in feet	10	10	$12\frac{1}{2}$	$14\frac{1}{2}$
Steam pressure, in pounds	90	90	160	180
Makers	Smit, Holland.	Smit, Holland.	Simons, Renfrew.	Simons, Renfrew.
Specified time for filling hopper, in minutes	25	40	40	30

The cost of working one of the largest dredgers at East London of, say, 900 tons hopper capacity, would be from £11 000 to £12 000 per annum, including an annual overhaul and insurance charges.

As illustrating the extent to which the depth in the entrance at East London has been improved since the works were started, it may be observed that when the late Sir John Coode first commenced work there in 1877, the depth in the entrance was only 5 ft. at low water, the rise of tide being likewise 5 ft. Now the entrance has been deepened to admit of vessels of 5 000 tons capacity regularly entering and leaving the port, this result having been brought about mainly by the introduction of pump dredging, started there in 1886, supplemented by the action of training and sheltering works, which tend to maintain the dredging but to a somewhat limited extent, owing to the inadequacy of the backwater and scour.

A pump dredger, in consequence of its special arrangements

with regard to swivelling suction pipes and otherwise, is adapted for working in a seaway where the use of a bucket dredger would be quite impracticable. For this reason also, it will be seen what an important influence pump dredging now has and will still more in the future exercise in the deepening of bar entrances.

No more successful application of this system can be referred to than that of deepening the bar and entrance to the Mersey, since the commencement of dredging there in 1890, at which date the depth on the bar was 11 ft. at low water of spring tide. This depth had been gradually increased by dredging to 27 ft. in 1900, which has since been fairly maintained. The pump dredgers used are the *Brancker* and *Crow*, each having a carrying capacity of 3 000 tons. The quantities removed by these dredges on the bar and in the channels of the Mersey for the year ending July 1st, 1904, and since the commencement of dredging operations, respectively, are shown in Table 2.

TABLE 2.

	During the year ending July 1st, 1904.	From commencement of dredging operations to July 1st, 1904.
	Tons.	Tons.
Queen's Channel Bar...........	1 450 300	30 930 640
Queen's Channel.....	161 700	18 768 620
Crosby Channel...	6 311 300	29 283 610
Total.....................	7 923 300	78 982 870

The cost of removal has been on the average about 1 penny per ton, exclusive of insurance, depreciation and interest on capital.

The writer is indebted to Mr. Anthony G. Lyster, the Engineer to the Mersey Docks and Harbour Board, for the foregoing particulars of the dredging at that port.

The conservation of the backwater in estuaries, in order to maintain the depth in the channels of the same and over bar entrances, is, in most cases, a matter of extreme importance. Although by the adoption of suitable training works, with a view to improve tidal development, reclamations and the abstraction of tidal water due thereto may be occasionally admissible, all such cases require most careful consideration prior to sanction.

The importance of the full maintenance of backwater, in keeping open harbour and river entrances, is well understood. An example of the benefit derived from scour due to backwater and tidal influence, is afforded by a comparison of the entrance at Yarmouth with that at Lowestoft on the east coast. Both these ports are subject to practically the same wave stroke and travel of sand and beach. There is, however, a considerable difference in the volume of the tidal compartment, and in the quantity of fresh water and tidal discharge. At Yarmouth there is a magnificent backwater with tributaries running into it, which results in an immense discharge of tidal and fresh water, more than sufficient to keep open the harbour without dredging; in fact, during the thirty years the writer has known this port, the entrance to the harbour and its approaches have not shoaled, but have deepened, there being, moreover, an active travel of sand and beach. At Lowestoft, a few miles away, the harbour is kept open with difficulty, by almost continuous dredging, due solely to the fact that the volume of tidal water is comparatively small, and insufficient for scouring purposes.

General Remarks on Harbour Construction.

The introduction of Portland cement, and, in later years, its extensive use in harbour construction, coupled with the development and perfection of plant and appliances used in the execution of sea works, have, to a considerable extent, revolutionized old methods of procedure, and undoubtedly tended to rapidity of construction, economy in cost, and in minimising sea risk during execution and subsequent thereto.

Lias lime, before the introduction of Portland cement, was the material principally used in mortar and concrete for sea works. In consequence of the time required for its setting, it is not a satisfactory material, especially in a seaway, where it is important that early setting, sufficient to secure the work and to prevent its disturbance by sea action, should be effected.

Latterly, a committee has been sitting in London, with a view to the preparation of a standard specification for Portland cement. This specification will shortly be issued, and it is hoped that an improved material, generally, will result therefrom. Uniformity of requirements and tests, whilst being of great advantage to the manufacturer, cannot fail to produce a more reliable cement, both with

regard to quality, strength and, that highly important requirement, soundness.

In late years the practice of bulking cement, after its delivery at the works for use, has considerably extended. It is no uncommon occurrence, on English works, on the arrival of the cement, to "shoot" it from the sacks into a heap 4 ft. thick, or thereabouts, in lots of from 200 to 250 tons, turning the material so bulked five times, at intervals of a week, the cement not being used in the works until after the expiration of five weeks from delivery. The material is cooled and aerated by this process, and its soundness generally assured. Sometimes, however, additional turnings are required, before the cement can be pronounced as sound. The increase of bulk, up to about five turns of the cement, is sufficient to pay the cost of such turning.

Other methods of treating cement are also adopted by English engineers, with a view to its aeration and the promotion of its soundness.

With reference to the use of cement concrete, in blocks, or in mass, for sea works, the writer's experience is generally in favour of the former. Sound work has unquestionably been produced by the use of mass concrete, but it is believed that failures have been more numerous, in consequence of the adoption of the material in this form, than from the use of concrete in blocks. Allusion is here made to solid structures and not to pocketed works, consisting of external walls with intervening cross-walls, the spaces being filled with rubble or other loose material, a form of work which is often attended with considerable risk, more particularly during execution.

In a tideway it is of the utmost importance, where mass concrete is to be used, that the relative sizes and proportions of the aggregate should be such as to produce an absolutely water-tight material. The infiltration of sea water, sometimes under considerable pressure, into "green" or unset concrete, and its subsequent exudation, which occurs twice in twenty-four hours, consequent on tidal action, causes the magnesium salts, in the water, to withdraw a portion of the lime of the cement in the form of calcium salts, and leave a deposit of magnesia in its place. It is this magnesia, derived from sea water, either alone or mixed with lime from the cement, which constitutes the white substance deposited in the interstices of porous

concrete between high and low-water levels. Concrete so affected possesses but little strength and its failure is only a question of time.

Given a fairly extensive work to construct, which will justify the cost of heavy special plant, the most economical mode of procedure is, generally, to use concrete in blocks, up to, say, 40 or even 50 tons in weight. Such blocks when laid in place, except under special conditions of exposure, to which reference will be made later on in this paper, would not be liable to subsequent disturbance, nor would concrete in this form, when fully set, or largely so, be subject to the action to which reference has already been made when referring to mass concrete.

In many works now being executed under English engineers, the concrete blocks, above low-water level, are faced with granite, or other suitable stone. This facing is bonded into the concrete at the time the block is moulded in the yard, and makes thoroughly good and water-tight work.

With reference to the general arrangement and lines of the works to constitute a harbour, everything turns on the local conditions and requirements. For instance, at Colombo, where the enclosed area is 660 acres, in consequence of a plentiful supply of cheap coolie labour, it is found that steamers obtain quicker and more economical transhipment and coaling, from barges, when lying out in the harbour, than would result from berthing at quays. Consequently no deep-water quay accommodation has been provided at that port. At the Commercial Harbour at Dover, and also at the new pier at Folkestone, the object to be attained was largely the provision of sheltered and convenient berthage for cross-channel and other steamers. At the Tyne the piers at the entrance are required for shelter and for concentration of scour. Reference is made to these cases in order to show how impracticable it would be to prepare rules which would be generally applicable in the arrangement and design of such works in other localities.

Rubble-Mound Breakwaters.

Probably two of the finest rubble-mound breakwaters in existence, are the outer breakwater at Portland and the breakwater at Table Bay, Capetown, each designed and carried out under the direction of the late Sir John Coode. Works of this class, in consequence of

the rolling of the rubble on the seaward slopes by wave action, require feeding with new material to compensate for the attrition of the stone and the wastage caused thereby. In most cases it is important to deposit the rubble from a temporary staging, so that the mound may be replenished, readily, with fresh material, from time to time, as the slopes are clawed down by the sea, until the normal angles of repose have been produced.

Works of this class are more adapted for positions where there is a small rise of tide, than where there is a considerable tidal range. At Portland, spring tides rise 6 ft. 9 in., at Table Bay, 5 ft. 0 in., and at Colombo, 2 ft. 0 in. At the latter the northeast breakwater consists of a rubble mound, and has afforded very satisfactory results.

It may be of interest to state that the final dimensions, and angles of the seaward slopes, assumed by the rubble at Table Bay Breakwater, which is exposed to a specially heavy wave stroke, are as follows:

Crest of mound, above L. W. O. S. T.	20 ft.
Width of mound at crest	30 ft.
Slope, crest to H. W. O. S. T.	2½ to 1
Slope, H. W. to L. W. O. S. T.	5 to 1
Slope, L. W. O. S. T. to 15 ft. below that level	7 to 1
Thence downward	1½ to 1
Breadth of mound at base	360 ft.

Composite Breakwaters.

These works consist of a built superstructure resting on a rubble base or mound. Holyhead Breakwater is a fine and successful example of this class of work, an account of which will be found in the *Minutes of Proceedings* of the Institution of Civil Engineers. Alderney Breakwater is also described in the *Minutes of Proceedings,* but may be regarded as a disastrous work, as will be gathered from a perusal of the record of it given by Mr. Vernon-Harcourt in his paper read before the Institution, and the discussion arising thereon.

The recoil or downward action of seas, striking the vertical face of the built portion of such structures, sometimes causes serious damage by scooping out the rubble near the footings and under-

mining the lowest courses of the work. It is, therefore, of great importance, where such action is anticipated, having regard to exposure and depth of foundations below low-water level, that the surface of the mound, adjacent to the sea footings of the work, should be effectively protected by a coating of specially heavy rubble, or by a wave breaker of pell-mell concrete blocks, or by an apron of concrete in bags or blocks.

Parapets along such structures generally intensify the recoil action alluded to, and, therefore, as a rule, should not be provided unless sufficient reason exists to render their adoption necessary, or that the sea stroke to be contended with is not of such weight as to cause their presence to be objectionable.

Breakwaters having a superstructure of concrete blocks arranged with sloping bond, or inclined slices, resting on a rubble mound, such as the works at Colombo and Madras, are perhaps deserving of special mention.

The works at Colombo consist of a southwest breakwater, 4 200 ft. in length, which was commenced in 1876 and completed in 1884. The northwest breakwater, an island work of 2 660 ft. in length, is drawing near completion, and a northeast breakwater of 1 100 ft. in length, to which reference has already been made, is almost finished. Between the terminations of the southwest and northwest breakwaters, there is an opening, or harbour entrance, 800 ft. in width, whilst between the northern end of the last-named work and the outer end of the northeast breakwater, there is another opening, 700 ft. width. The enclosed area embraced by these three works is 660 acres. The depth, generally, over this area is from 30 to 38 ft. at low water, the rise of tide being 2 ft.

The concrete blockwork of the southwest breakwater is 34 ft. in width, and is founded on a rubble base consisting of gneiss rock deposited from barges at a depth of 20 ft. below low water. The northwest breakwater is somewhat less exposed, and the blockwork is 32 ft. in width, but being constructed at a later date, it was considered prudent to found it at a lower level than the previous structure, so that the foundation blocks, throughout this work, are laid at an uniform depth of 30 ft. 9 in. below low water.

The blocks in each of the two breakwaters are set with sloping bond, in slices of 5 ft. 6 in. in width, the weight of the blocks varying from 33 to 17 tons, according to their position in the work,

and the bond which is to be produced. The top of the block-work when laid is 8 ft. above low-water level in the south-west breakwater and 7 ft. in the northwest breakwater, on which there is a capping of concrete in mass, extending practically over the full width of each structure. The capping is 11 ft. 6 in. above low water in the southwest breakwater, and will be 10 ft. 6 in. above that level in the northwest breakwater when it is fully completed. There is no parapet on either structure.

In addition to the bonding of the blocks, joggles—five in number—in each slice, are formed of concrete bags, extending from the top to the bottom blocks, the bags being thus deposited in cavities formed half in each slice, so that when the slices come together, oblong spaces are produced for the reception of the bags, 18 in. long and 12 in. wide, extending from the top of the foundation course to the surface of the blockwork.

The blocks and capping throughout are of 6 to 1 concrete and the joggles of 5 to 1, the latter being deposited in jute bags. The mound, in the case of the old or southwest breakwater, was protected on the sea side adjacent to the footings, by an apron of concrete in bags, 22 ft. in width and 2 ft. 6 in. thick, each bag being of 12 tons in weight. In the new or northwest work, it has been found sufficient, in consequence of the increased depth of the foundations, to protect the footings by a deposit of selected heavy rubble stone on the mound, after the blocks have been laid.

The concrete blocks in each of the two main works, were set from a Titan crane, specially designed to meet the peculiar conditions of each breakwater. These machines travelled along the works as set.

At Colombo there are two monsoons, covering the entire year, *viz.*, the southwest monsoon, lasting from the end of May to the end of October, and the northeast monsoon, occupying the remaining portion of the year. During the last-named monsoon the weather is fine and the sea comparatively smooth, so that setting operations on the breakwaters are confined to these periods. In the six months when work was practicable, the southwest breakwater was extended as much as 950 ft., and the northwest breakwater, which is an island structure, 650 ft.

It may be of interest to observe that the whole of the operations

in the quarries in connection with these large works at Colombo, have been and still are carried on by convict labour, with the exception of the foremen and the men engaged in firing the explosives. Upwards of 700 convicts are employed in these quarries, and in certain minor operations in other portions of the works.

It may likewise be mentioned that, for a large graving dock now in course of construction at Colombo, gneiss stones are being dressed, in a perfect manner, by native labour, and by men hitherto unacquainted with such work, they having been instructed by qualified masons sent from England for that object. Similarly, gneiss copings, steps and other work are likewise carefully dressed by convict labour, under corresponding instruction.

The two Madras breakwaters were likewise formed of sloping blocks on a rubble base, but instead of the courses being bonded and tied together with concrete joggles, as described, a vertical joint was formed along the center throughout the entire length of each work, with the object of providing for unequal settlement in the mound. As a result, however, on the occasion of a specially heavy cyclone, great damage was done by the sea breaking over the works, carrying off the uppermost blocks on the inner half of the structure and subsequently demolishing the outer portion of the fabric along considerable lengths.

A capping of concrete in mass was not provided at these works, as was done at Colombo, which capping tied the structure together, and prevented the sea action which was found to be so prejudicial at Madras.

The Madras breakwaters have now been made good, and a wave breaker of pell-mell concrete blocks provided, to assist in dissipating the unusually heavy seas to which these works are exposed, owing to their falling within the zone of cyclonic influence.

As a further instance of a structure coming under the description of a composite breakwater, allusion may be made to the north pier at the entrance to the Tyne. This pier was commenced in 1854, and is 2 960 ft. in length. The inner portion consists of pocketed work, founded on a rubble base about 2 ft. below low water. The outer length is solid, and the foundations, which are also on rubble, extend from 16½ to 27 ft. below low water, the latter including the foundations around the head.

In January, 1897, a breach was formed in the outer length of the work, which was subsequently extended by the action of the sea, until it attained a length of 310 ft., right across the entire breadth of the structure, the debris reaching up to about half-tide level. The original work, where this breach occurred, was founded at from 21 to 24 ft. below low water, the rise of tide being 15 ft.

The disturbance of the mound, in front of and adjacent to the breached portion, occurred where its surface was protected against the recoil action of the sea by an apron consisting of two rows of concrete blocks, each of 41 tons in weight, which blocks were removed by sea action, and the work subsequently undermined.

After investigation, and on tenders being obtained for the repair of the damage, it was found, in consequence of the tremendous seas rushing through the breach, and from other causes, that, by incurring an expenditure but little in excess of the sum required to make good the work along the line of the breach, and on either side thereof where shaken, a length of 1 500 ft. of new breakwater could be provided under the lee of the old structure, using the latter as a shelter. Consequently, after due consideration, this mode of procedure was sanctioned, and is now in course of active accomplishment. The new work is founded at a level of 44 ft., or thereabouts, below low water, on the natural shale and marl bed, which extends along the sea bottom where the structure is being formed.

Reference has been made in the early portion of this paper, to the observed height of the waves in the vicinity of these works.

With a view to preventing lateral movement of the blocks, in the new structure, and also in the breakwater at Peterhead, in consequence of the abnormally heavy wave stroke to be provided for, arrangements have been made to form a continuous check, or projection of 6 in. in height in each course, from low water downward, in the vicinity of the center line, dependent on the bond, thus raising the inner half, or harbour side, of each course, 6 in. above the seaward portion, and thereby creating a barrier to prevent movement shoreward on the occurrence of excessive gales. This system was introduced by the writer's firm about three years ago, and it is believed, from subsequent experience, that it will prove effective for the attainment of the object in view.

Above the low-water course the blocks are set and grouted in

Portland cement, and, being likewise bonded and joggled, are, with respect to this portion, practically monolithic.

Breakwaters and Piers of Built Concrete Blocks.

Brief reference may here be made to the works recently completed and in progress at Dover. The Prince of Wales Pier was carried out for the Dover Harbour Board, and is a structure 2 900 ft. in length, *viz.*, 1 250 ft. of viaduct at the inner end for tidal circulation, and 1 650 ft. of solid work at the outer end for shelter and berthage.

This work was executed under a contract with Sir John Jackson, who is entitled, the writer considers, to the credit of the initiation of an improved form of temporary staging, which has since been adopted in connection with the Admiralty Harbour works, and at Folkestone, and likewise for the development of the bell system of under-water work, by the introduction of bells of a much larger and more perfect character than had hitherto been used.

In the construction of the Admiralty Harbour Works at Dover, for which Messrs. Pearson and Son are the contractors, further and considerable developments and improvements have been introduced, both with regard to the temporary staging and the bells, and there is reason to believe that no sea works have hitherto been constructed, where plant and appliances, generally, of such perfect character, as are in use on these large works, have been adopted.

The further works contemplated by the Dover Harbour Board, comprise the construction of a spur pier from the Admiralty Pier extension, in order to enclose the Commercial Harbour, the latter having an area of 71 acres, and also the widening of the Prince of Wales Pier with a view to the formation of a large water station, with berthage for cross-channel steamers on the western side, and for American and other liners on the east face.

The National Harbour works at Dover, entail the construction of nearly 2 miles of deep-sea breakwaters and piers, in addition to a sea-wall, 3 500 ft. in length, facing a reclaimed area of 21 acres.

The sea structures throughout are formed of concrete blocks, each from 40 to 26 tons in weight, dependent upon the arrangement of the bond in the work. The average weight of the blocks throughout is 32½ tons, and the concrete used in their construction 6 to 1.

Each work is faced with granite built into the blocks in the yard, from and including the low-water course upward, in the manner explained, and all the blocks from foundation level are bonded and joggled. Above low water, the courses are bedded and grouted in cement.

There will be two entrances to the main harbour, the westernmost one being 800 ft. in width and the easternmost 600 ft. Arrangements have been made for the adjustment of each of these openings, as experience may show to be desirable, when the works draw toward each other during construction.

The total area of the National Harbour is 610 acres, which, added to the Commercial Harbour of 71 acres, forms a total area of sheltered water enclosed by the main works of 681 acres. The whole of the National, as well as the Harbour Board works, are founded on a sound and compact bed of chalk, or flints embedded in chalk.

The blocks in each of the structures have been set from travelling Goliath cranes running on a temporary staging. In the case of the Admiralty works the roads on the stagings are 27 ft. 6 in. above high water of spring tides, which tides rise 18 ft. 9 in. The piles are arranged in trestles or clusters of six, to form spans of 50 ft. 3 in., and the Goliaths, both on the stagings and in the workyards, are all of 100-ft. gauge, and are, unquestionably, the largest and most powerful machines of the kind hitherto used.

There is a marked advantage in the adoption of the system of stagings having Goliath cranes thereon, over the Titan method of setting, for such works as those at Dover, keeping in view the great height of the structures from foundations to formation level, and the impracticability, under the Titan system, of carrying on simultaneously, the several operations which are necessary at Dover, for the attainment of reasonable progress.

In illustration it may be remarked that on each of the temporary stagings at Dover, the following plant is used: At the outer end a stage-erecting machine, followed by a travelling Goliath crane working a grab; then a second Goliath from which the diving-bell is worked. Following this latter machine comes a third Goliath for block-setting under water, and behind again, a fourth Goliath for setting blocks above low-water level, and if not required for this purpose, then for supplying blocks to the third Goliath.

It will thus be seen that five distinct operations may be carried on under this system, at the same time, with perfect convenience and facility, whereas with the Titan system only one process is available, unless other means are adopted for preparing and levelling the foundations in advance, which but seldom admits of practical accomplishment. Therefore, generally speaking, except for the setting of sloping block structures such as those at Colombo, where the scar end projects but little and the height of the blockwork is not excessive, more rapid execution is practicable, and greater facilities are afforded by the staging and traveller system, than with the Titan method of setting.

As illustrating the magnitude of the works at Dover, it may be mentioned that the height of the Admiralty Pier extension (which is 2 000 ft. in length) from foundation level to top of parapet, is over 90 ft., the depth alongside at low water being 42 ft., exclusive of the sinking of the foundations below the surface of the natural sea bed, the rise of tide being 18 ft. 9 in.

The rate of progress with the East Arm foundations for 1901, 1902 and 1903, has been practically 1 000 ft. per annum, to compare with an average progress in the construction of the old Admiralty Pier, between 1847 and 1871, of 91 ft. per annum, this result being brought about by the use of large blocks, of 32½ tons weight on the average, as compared with 8 tons in the old structure, and the more perfect plant, staging and appliances which are now available to which reference has been made.

The diving bells used on the National works are 17 ft. 6 in. by 10 ft. 0 in. by 6 ft. 6 in. headroom. They weigh, out of the water, about 35 tons, or 5 tons when submerged and in work. They are fitted with the electric light and telephonic communication with the top, but mechanical signals are preferred by the men.

The foundations generally are carried from 4 to 6 ft. into the chalk and flint bed, and are protected on their seaward sides from abrading action, due to the running of seas along the back of the work, by an apron of concrete blocks, 25 ft. in width.

The contract provides that grabbing of the material along the lines of the works may be adopted by the contractor down to within 12 in. of the finished level of the foundations, the latter is, however, to be removed by bell divers. Hence the adoption of the fine bell

arrangements, both on these works and those previously carried out for the Dover Harbour Board.

Four men are engaged in the bell excavating and finishing the foundations ready to receive the lowest course of blocks. Each shift is of three hours' duration, and two shifts *per diem* are generally worked by the same men. The bells are in use night and day when the weather is favourable.

The greatest depth of the foundations on these works is 53 ft. below L. W. O.S.T., the average depth being 47 ft. below that level. There is thus an average working head of 66 ft. at H. W. O. S. T., corresponding to a pressure of 29 lb. This has been found to be as deep as the men can comfortably work in the bell, and, on some occasions when this depth has been exceeded for a short period, inconvenience has been experienced in consequence of the extra pressure.

Very perfect plant has also been provided in the workyards in connection with the preparation of the concrete blocks. In the yard of the east section, where the blocks have been, and are being, made for the East Arm and South Breakwater, six electric portable concrete mixers are used, each capable of turning out about 100 cu. yd. of concrete *per diem*. These mixers are of the Messent type, revolved by a motor of 18 h. p., and travelled, from the point where the aggregate is received, to the block moulds where the finished concrete is deposited, by a 25 h-p. motor. The gauge of these electric mixers is 11 ft. 7 in., and their use has been attended with most satisfactory results.

Notwithstanding the alarming reports which have appeared in the press, from time to time, with regard to damage to the works at Dover, it is satisfactory to state that practically no damage whatever, from the first, has been occasioned to the permanent works, and only comparatively insignificant damage, having regard to the magnitude of the undertaking, has been caused to the temporary structures. Although it was alleged, that during the great gale in September, 1903, 1 000 ft. of breakwater works and staging had been carried away, the only loss which was occasioned was the turning over of one span of temporary staging of 50 ft. in length with the plant thereon, which at that time occupied an isolated position.

The blocks for the Prince of Wales Pier had a maximum weight of 20 tons, and were faced above low water with granite blocker courses of less average thickness than those adopted for the heavier and more exposed breakwaters, in connection with the National Harbour.

It has only been practicable, in the foregoing paper, to touch on the salient features of the works referred to. There are other types of breakwaters and piers, such as structures of concrete in mass deposited within piling and temporary shuttering and structures of concrete in bags up to low-water level, carrying a solid work built of concrete blocks, such as the fine pier at Roker recently completed at the entrance to the Wear at Sunderland. Space does not, however, admit of more than a bare reference to these further works.

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HARBORS.

HARBOUR DEVELOPMENT IN HOLLAND.

By H. Wortman.*

The North Sea along the coast of Holland being very shallow, the harbours of the country, in earlier times, were established along the various branches of the Rivers Rhine, Maas and Schelde, which have their outlets on this part of the coast, and along the great inland sea, called "Zuyder Zee." (See Fig. 15.)

Harbour of Ymuiden.

The first harbour, directly facing the North Sea, was built in 1867–77 at the mouth of the North Sea Canal, which was constructed at that time in order to form a new and direct way for sea-going vessels to the Port of Amsterdam, till then only accessible by the Zuyder Zee and by a long, narrow canal, called "Noordhollandsch Kanaal," leading to the Harbour of Helder.

At Ymuiden two breakwaters were constructed, Fig. 16, with a length of 1 528 m. (5 000 ft.) each, enclosing an outer harbour with an area of nearly 120 hectares (300 acres) at high water, in which a channel was dredged to a depth of from 7.50 to 8.50 m.

* Engr., Royal Corps of Waterstaat, Amsterdam, The Netherlands.

(24 ft. 7 in. to 27 ft. 11 in.) below A. P.,* the ordinary low-water level at Ymuiden being — 0.79 m. (2 ft. 7 in.) A. P., and the ordinary high-water level + 0.74 m. (2 ft. 5 in.) A. P.

The breakwaters, Fig. 16, were of concrete blocks set by a Titan in horizontal layers on a rubble mound, and covered with a monolithic mass of concrete. After the completion of the work the breakwaters were protected on the sea face by a mound of concrete blocks of from 5 to 10 cu. m. (6½ to 13 cu. yd.), deposited at random by a derrick crane moving on the breakwater.

As some of the protecting blocks, during each heavy gale, are damaged and thrown aside by the action of the waves, the block-mounds have to be repaired at regular intervals by depositing new concrete blocks composed of 2 parts of Portland cement, 3 parts of sand and 5 parts of gravel, at an average rate of 30 blocks a year.

The concrete of the superstructure, composed of 1 part of Portland cement, 3 parts of sand and 5 parts of gravel, having been damaged at various places by the action of the sea water, a facing of brick masonry has been applied to it above the high-water level.

This repair work and the placing of new protecting blocks has been done since 1880 at an average annual expense of about 35 000 guilders ($14 000).

The channel in the outer harbour gives access to the sea locks of the North Sea Canal at Ymuiden.

As a new sea lock had been constructed at Ymuiden in 1891–95, with a length of 282 m. (925 ft.), a width of 25 m. (82 ft.), and a depth of 10.15 m. (33 ft. 4 in.) below A. P., in order to make the North Sea Canal and the Port of Amsterdam accessible for larger vessels, the depth of the channel in the outer harbour has been increased since by dredging to — 10.50 m. (34½ ft.) A. P., the depth at the mouth of the harbour being now about —12 m. (39½ ft.) A. P.

To maintain the depth in the Harbour of Ymuiden, an average quantity of about 600 000 cu. m. (785 000 cu. yd.) of sand and silt, deposited in it by the sea, has to be dredged yearly, at an expense of nearly 150 000 guilders ($60 000).

The dredging is done by self-loading suction dredges and by bucket dredges, the sand being carried seaward and unloaded at a depth of 15 m. (49 ft. 3 in.) and more.

* A. P., *Amsterdamsche Peil*, is the datum accepted for all leveling in Holland.

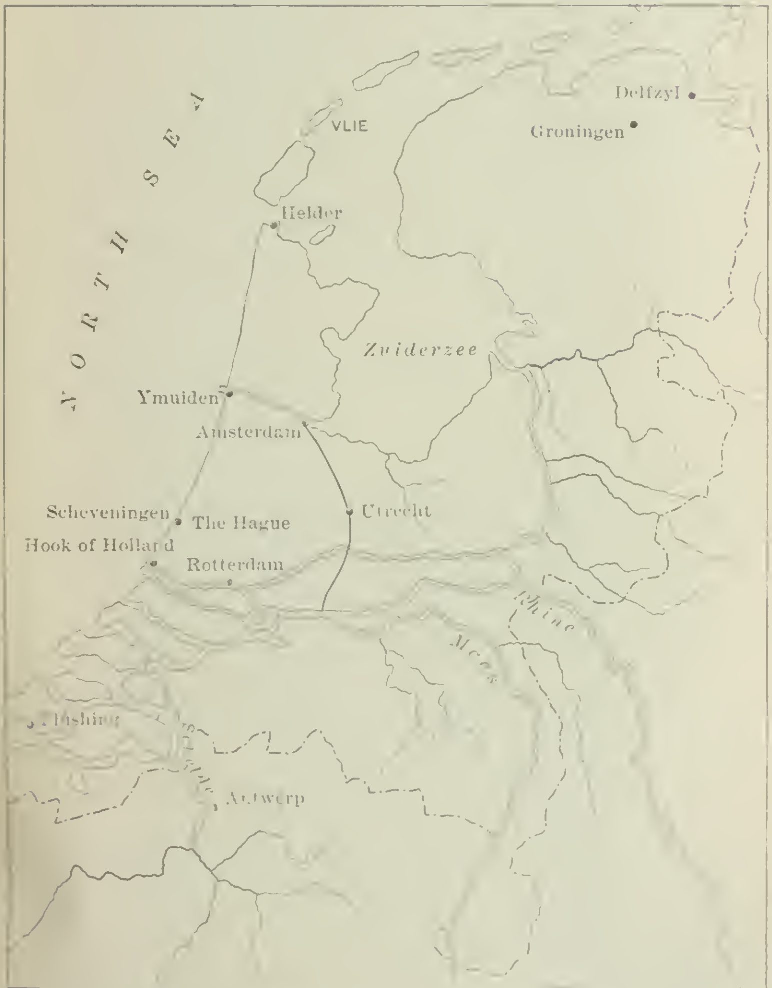

NORTH SEA
VLIE
Delfzyl
Groningen
Helder
Zuiderzee
Ymuiden
Amsterdam
Utrecht
Scheveningen
The Hague
Hook of Holland
Rotterdam
Rhine
Meuse
Antwerp

FIG. 15.

The North Sea Canal and the Harbour of Ymuiden were built by the Amsterdam Canal Company, with contributions from the State and the City of Amsterdam. In 1883, after the completion of the works, they were transferred to the Government and since then they have been maintained at the expense of the State.

Fishing Harbour at Ymuiden.

As soon as the harbour at Ymuiden and the North Sea Canal to Amsterdam were completed, the latter ending eastward from Amsterdam in the Zuyder Zee and separated from it by locks, the fishing boats of the Zuyder Zee, which formerly went fishing to the North Sea by the Passes of Helder and Vlie, commenced to use the new canal and to seek shelter on stormy days in the outer harbour at Ymuiden.

The number of these vessels increasing steadily, they presented at length a serious obstacle to the entrance of the sea locks. It was resolved, therefore, that a harbour for fishing boats should be constructed by the Government at Ymuiden, at the outer side of the sea locks. This fishing harbour, Fig. 16, was constructed in 1894–97. It has an area of 6.5 hectares (16 acres) and a depth of 5.80 m. below A. P., or 5.01 m. (16 ft. 5 in.), below ordinary low water, and is furnished with a quay wall, Fig. 16, on one side and a wooden landing stage on the opposite side, a fish-hall serving for the sale of the fish. The opening of this harbour in 1897 led to an important development of the fisheries and fish trade at Ymuiden. The number of vessels visiting the harbour has increased from 9 794 in 1897 to 14 310 in 1903, among which the number of steam trawlers increased from 84 in 1897 to 2 057 in 1903, 51 of these belonging to Ymuiden ship owners.

On account of this development of traffic, the extension of the harbour proved necessary very soon after the opening; this was effected in 1903, the area of water being increased to 9 hectares (22 acres) and the depth to 6 m. (19 ft. 8 in.) below A. P., while still further extension has been projected.

Landing Stage.—The extension of the harbour in 1903 made necessary the construction of a quay wall, or landing stage, along the southwest side of the new basin, in order to load coal and ice directly from the railway into the vessels.

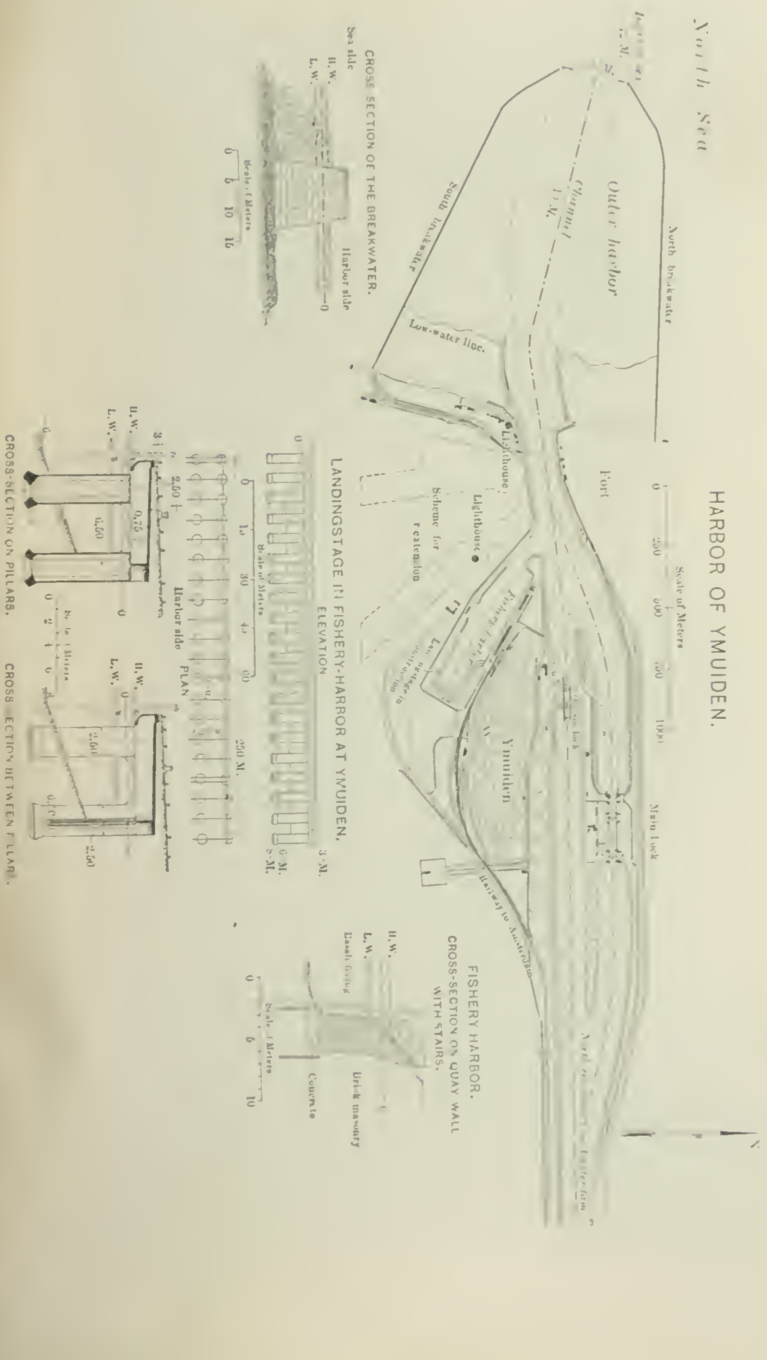

HARBOR OF YMUIDEN.
North Sea
North breakwater
South breakwater
Outer harbor
Channel
Low-water line.
Fort
Lighthouse
Scheme for extension
Ymuiden
Main lock
Railway to Amsterdam
Scale of Meters
0 250 500 750 1000
CROSS SECTION OF THE BREAKWATER.
Sea side
Harbor side
H.W.
L.W.
Scale of Meters
0 5 10 15
LANDINGSTAGE IN FISHERY-HARBOR AT YMUIDEN.
ELEVATION
PLAN
Harbor side
Scale of Meters
0 15 30 45 60
250 M.
CROSS-SECTION ON PILLARS.
CROSS-SECTION BETWEEN PILLARS.
FISHERY HARBOR.
CROSS-SECTION ON QUAY WALL WITH STAIRS.
H.W.
L.W.
Brick masonry
Concrete
Scale of Meters
0 5 10

FIG. 16.

It was decided that a landing stage, Fig. 16, should be constructed of concrete and steel. This work is in course of construction under the direction of the writer, after the design of Mr. L. Sanders, Engineer of the Amsterdam Company for Concrete and Steel Construction. As the principle of Mr. Sanders' scheme is a new one, it may be interesting to give some details of this work.

Instead of choosing a system of piles and girders of concrete and steel, as Mr. Hennebique has done in many similar cases (Southampton, Nantes and other places along the French coast), Mr. Sanders rests his girders and deck on a double range of hollow pillars, which are moulded of concrete and steel on neighbouring ground, and afterward carried by a traveler, Fig. 1, Plate IV, or by a floating jib, Fig. 2, Plate IV, to the spot where they are to be sunk. The pillars have a height of 8.75 m. (28 ft. 8 in.) and a diameter of 2.50 m. (8 ft. 2½ in.); they are placed in each range at a distance of 5 m. (16 ft. 5 in.) from center to center, the two ranges being 6.5 m. (21 ft. 4 in.) from center to center.

The sinking of the wells, which are placed partly in the sand and partly in the water of the completed portion of the harbour (by the floating jib), is done in the following manner:

At first an additional cylinder, 3.75 m. (12 ft. 4 in.) wide, made also of reinforced concrete and provided with a steel edge, is sunk in the sand by water-jets, supplied by fifty injection tubes, ranged along the outer side of the cylinder, Plate V, the water being pumped under a pressure of about three atmospheres by two pulsometers and three special pumps. The additional cylinder by the traveler, or by the floating jib, placed in the cylinder and having been sunk to the proposed depth of —8 m. A. P., the sand in it is removed by a sand-pump, then the pillar is carried out filled with sand, after which the cylinder is withdrawn, to be used elsewhere. The sinking of the pillars has met with no serious difficulties, as the soil at Ymuiden consists of sand with a few thin layers of clay. As an average, one pillar per day is sunk; sometimes, however, when a thicker layer of clay or some other obstacle is met, nearly a week has been spent in sinking one pillar.

Between the pillars of the back range, large plates of reinforced concrete are injected in the sand to the depth of 6.5 m. below

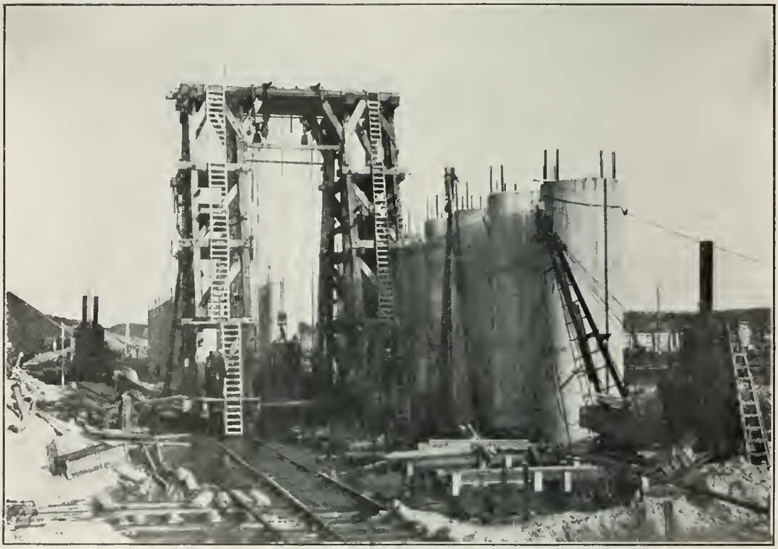


Fig. 1.—Pillars for Landing Stage at Ymuiden.

Fig. 2.—Pillar for Landing Stage Carried by Floating Jib.

A. P., thus forming a wall, to prevent the sand behind the landing stage from sliding. After tho sinking of these plates, the girders and deck-plates of reinforced concrete are moulded on the spot, the deck-plates are covered with a layer of sand in which the cross-ties of the railways are embedded, and then the basin along the work is dredged out to a depth of 6 m. below A. P.

Wooden pilework will be placed afterward in front of the landing stage to protect the concrete from being damaged by mooring vessels. The landing stage will be provided with two railway lines, and is designed to carry locomotives of 45 tons weight. A length of 250 m. (820 ft.) will be completed in 1904. The work is being done by the Amsterdam Company for Concrete and Steel Construction and Mr. van Hattum, as contractors, at an expense of 190 000 guilders ($76 000), or 760 guilders per m. ($278 per yd.), while a quay wall of brick masonry of the ordinary construction on a foundation of concrete, under the same conditions, would have cost about 1 100 guilders per m. ($481 per yd.).

Fishing Harbour at Scheveningen.

Another fishing harbour, Fig. 17, is constructed on the coast of the North Sea at Scheveningen, near The Hague. Scheveningen is the seat of very important herring fisheries. In early times the fishing boats, called "pinken" or "bommen," which have a draft of about 2 m. (6½ ft.) when fully loaded, were stranded on the shore, which is flat and sandy.

The coast being very much exposed to western winds, which prevail in Holland, this method of stranding became more and more dangerous, as the sandy coast was attacked at various times by the action of the waves during northwestern gales, and had to be protected by groynes or strand cribs. In 1894 and 1895 the Scheveningen fleet, lying on the shore during the winter season, was seriously damaged by strong gales. It was decided, therefore, in 1899, that a harbour should be constructed on the coast to secure a sheltered landing place for the fishing boats. The works were carried out in 1900–04, at a total expense of 2 500 000 guilders ($1 000 000), and consist of an outer harbour with a pair of breakwaters on the coast, constructed by the Government, and two inner

basins, excavated in the dunes by the City of The Hague, to which the Village of Scheveningen belongs.

Inner Harbour.—The inner harbour has two basins, the inner one being intended to receive vessels that come to unload their cargo, or to seek a shelter during the winter season, and the outer one offering place for stopping the speed of sailing vessels that enter the outer harbour from sea, and for preparing the departure of sailing vessels going to sea. The inner basin has an area of 6.8 hectares (16¾ acres), and is provided with quay walls and a landing stage. The outer basin presents at half-tide a water area of nearly 3.5 hectares (8½ acres), and is provided partly with quay walls and partly with stone-covered slopes. A dockyard at the southwest side of the outer basin is intended for the repair of fishing boats. The entrance of the outer basin is 60 m. (197 ft.) wide, the passage between the outer and inner basins is 40 m. (131 ft. 2 in.) wide.

The inner harbour is nearly completed. The works have been carried out by the City of The Hague, at an expense of nearly 1 600 000 guilders ($640 000), under the direction of Mr. van Voorst Vader.

About 3 000 000 cu. m. (492 000 000 cu. yd.) of sand had to be excavated to an average depth of 9 m. (29½ ft.), which was done with two excavators, the sand being carried off by railway to a neighbouring place amidst the dunes, at a distance of 3 km. (1.86 miles). As the water level in the subsoil at the commencement of the work was + 1.35 A. P., and the sand had to be excavated to —2.66 A. P., a draining plant was established, so as to lower the water level to the desired depth. The cost of excavating and transporting the sand, draining plant included, has been about 0.18 guilder per cu. m. ($0.055 per cu. yd.).

Outer Harbour.—The outer harbour, protected by two breakwaters, has an area of 6.5 hectares (16 acres). The entrance from the sea is 130 m. (426 ft.) wide, and is situated at a distance of 284 m. (931 ft.) seaward from the entrance of the inner harbour.

The depth to be given to Scheveningen Harbour has been the subject of many deliberations. Most of the passes for sea-going vessels which lead to the heart of the country being defended by forts, the War Department, when the construction of a harbour on

FISHERY-HARBOUR AT SCHEVENINGEN.

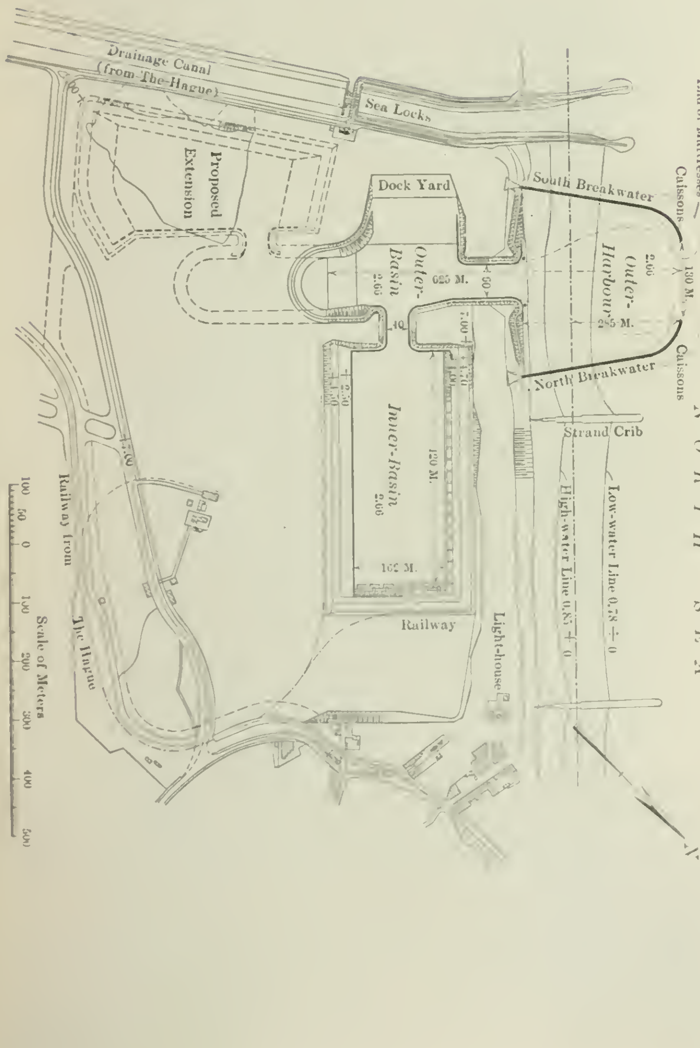


FIG. 17.

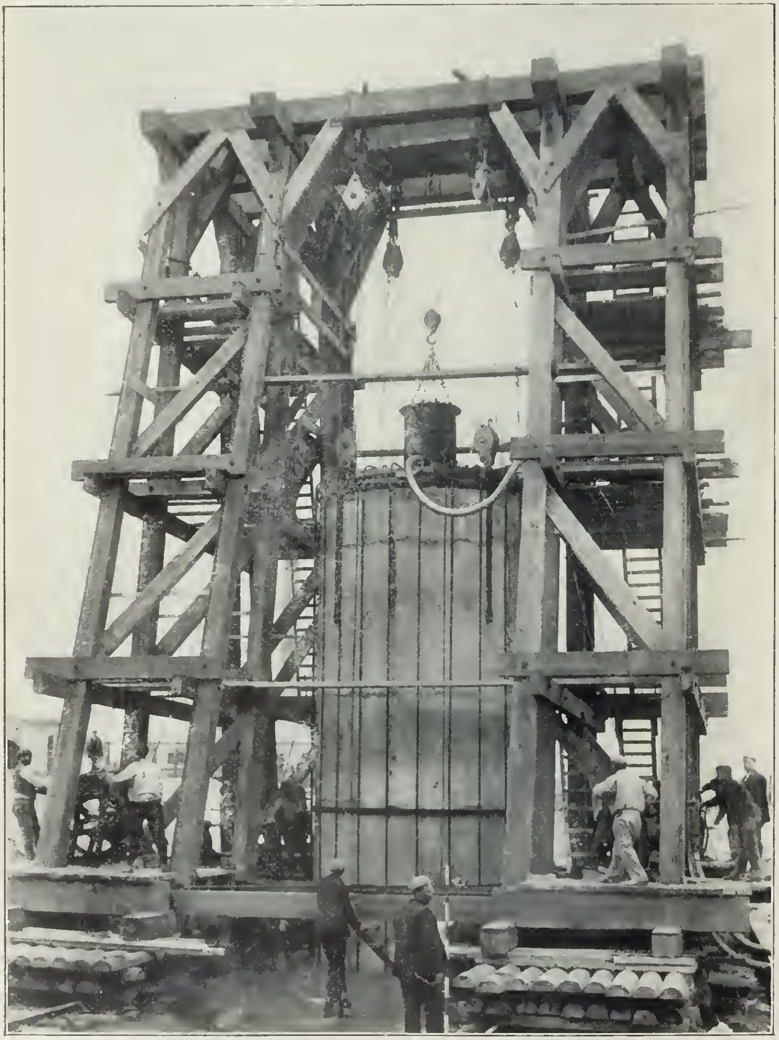
the coast at Scheveningen was planned, claimed the right to erect a fort to defend the entrance of the new harbour. As this would increase the cost of the whole by nearly 3 000 000 guilders, negotiations were opened, which resulted in the War Department allowing a harbour to be made without a fort, if its depth did not exceed —2.66 m. (8 ft. 9 in.) A. P. Therefore the depth of the harbour was fixed at that level.

As ordinary high water at Scheveningen rises to +0.85 m. A. P., and ordinary low water falls to —0.79 m. A. P., the depth in the harbour varies between 3.51 m. (11½ ft.) at high water and 1.87 m. (6 ft. 2 in.) at low water. The lowest ebb-tide noticed at Scheveningen has been —1.80 m. (5 ft. 11 in.) A. P., and the highest flood-tide, noticed during a northwestern gale, has been +3.53 m. (11 ft. 7 in.) A. P. Vessels with a draft of 2 m. (6 ft. 7 in.), a draft which most of the Scheveningen herring boats have when fully loaded, cannot enter or leave the harbour at low water, and must wait until the water has risen sufficiently.

On account of the depth allowed for the harbour, the breakwaters had to be carried out to the line where there was a depth of —3 m. (9 ft. 11 in.) A. P. At the time the works were to be executed, this depth proved to have been increased to —4 m. (13 ft. 1 in.) A. P., at which depth the pier-heads finally were founded. The construction of the breakwaters was carried out, from the writer's design and under his direction, in 1901–03, and was a difficult work, as the situation was very much exposed to western winds, which prevail in the North Sea, and there was no harbour in the neighbourhood which could serve as a port of refuge for the plant used in the construction, the nearest harbour being that of Hoek van Holland, at the mouth of the Waterway to Rotterdam, at a distance of 17 km. (10½ miles) from Scheveningen.

The mouth of the drainage canal of The Hague, which has its outlet into the sea southward from the new harbour, could only be used for vessels of very small draft, the channel being almost dry at low water.

As the Scheveningen "bommen" are known to be rather bad sailors, it was thought desirable to give a considerable width to the entrance between the pier-heads, which was fixed at 130 m. (426 ft.). In view of this width and of the short distance between the pier-



Sinking a Cylinder at Ymuiden.

heads and the entrance to the inner harbour, it was considered necessary that the breakwaters should extend above the highest waves of the sea so as to secure as quiet water as possible in the inner harbour. The height of the breakwaters was fixed, therefore, at +4.60 m. (15 ft. 1 in.) A. P., with a parapet extending to +5.60 m. (18 ft. 4½ in.) A. P. on the sea face.

It was decided that the breakwaters should be constructed of concrete blocks which were to be moulded at a sheltered place amidst the dunes, and brought to the work by railway.

As the bottom on which the breakwaters were to be built consists of sand, easily washed away by currents and wave action, it was necessary, in order to prevent the blocks from being undermined, to cover the sea bottom with a protecting layer. For this purpose it was covered with mattresses of fascine-work, which were made on shore, then floated to the spot at high water, and sunk by loading them with rubble (Fig. 1, Plate VI). These mattresses (called "zinkstukken") are generally used in Holland to protect shores and bottoms of rivers from being washed out by currents.

On this protecting layer of mattresses and rubble, about 0.50 m. (1 ft. 8 in.) thick and extending from 10 to 30 m. (33 to 98 ft.) beyond the sides of the breakwater, the blocks of concrete were to be set on a thin layer of gravel which filled up all the holes left between the rubble.

Concrete Blocks.—For the placing of the concrete blocks the system of inclined blocks was chosen. This system was first applied when building the Manora Breakwater at Karachee, India, and afterward at Colombo, also for some other breakwaters in India and Europe. According to this system, the blocks are set with an inclination of about 71° (1 to 3), partly covering each other, and each block preventing, by its weight, the blocks behind it from falling to one side (Fig. 18).

In the case of the Scheveningen breakwaters, it seemed particularly desirable to choose this system, in which each block is allowed to settle independently, as the layer of fascine and rubble under the blocks could give way to some settlement under the weight of the blocks and of the Titan and locomotives, moving upon the completed portion of the breakwater.

At Scheveningen the blocks have been set in three ranges,

Fig. 18, the blocks of the middle range being in advance of those of the side ranges by more than half their thickness, say 0.75 m. (2½ ft.), in order to avoid continuous transverse joints in the breakwater. The blocks of the middle range are 1 m. (3 ft. 4 in.) broad, and those of the side ranges from 2 to 2.50 m. (6 ft. 7 in. to 8 ft. 2 in.), according to the increasing breadth of the breakwater toward the end. All the blocks are 1.50 m. (4 ft. 11 in.) thick in the direction of the axis of the breakwater. Their height varies from 2.50 m. (8 ft. 2 in.) to 3.50 m. (11 ft. 6 in.), according to the increasing depth of the sea bottom toward the ends of the breakwater. Where the depth was 2 m. and more below A. P., two blocks were set upon each other, as the length of the blocks could not exceed 3.50 m., according to the maximum weight to be given to the blocks, which was fixed at 27 000 kg. (26½ tons). The blocks of the side ranges are tied together and fastened to those of the middle range by strong iron bars. These are fastened at the upper side in one of the holes with which the block was pierced for the passage of the bars serving to raise and place it in position.

The system adopted at Scheveningen has proved quite satisfactory, no difficulty having occurred during the setting of the blocks or afterward. The blocks were set by a Titan, Fig. 1, Plate VII, moving on the completed portion of the work. The Titan was of the revolving type, and could raise and place blocks of 27 000 kg. at a distance of 7.50 m. (24 ft. 7 in.) from its center, and blocks of 10 000 kg. at 9.50 m. (31 ft. 2 in.) from its center. It was built by MM. De Jongh et Cie., at Oudewater, Holland, and cost about 20 000 guilders ($8 000). It has done its work very well, not a single accident having occurred during the two working years.

A diver was always on hand to level the foundation of rubble and gravel on which the blocks were to be placed, and more gravel was spread under the block if necessary.

On an average, from 12 to 18 blocks were placed in 24 hours, the record having been 36 blocks. Nevertheless, the construction of the breakwaters has taken a long time, as the block setting was interrupted by bad weather and rough seas, the years 1902 and 1903 being very unfavourable for this kind of work.

On their upper side, the blocks reach about 1 m. (3 ft. 4 in.) above the ordinary high-water level. The superstructure of the

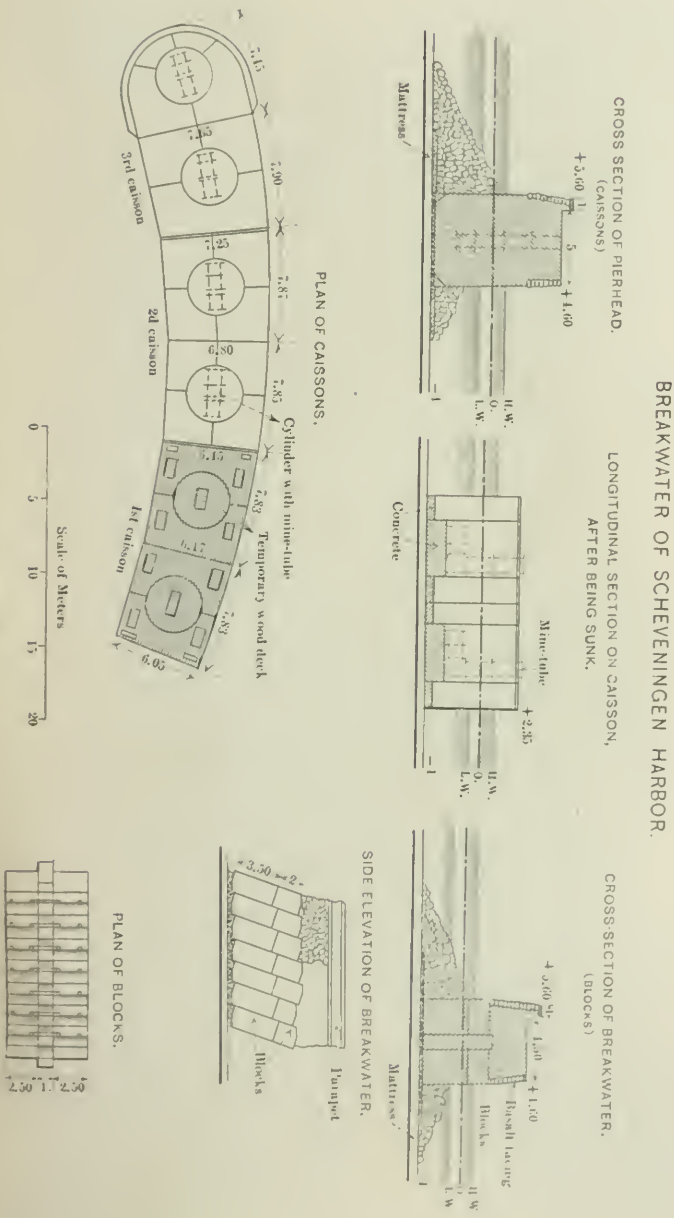

BREAKWATER OF SCHEVENINGEN HARBOR.
CROSS SECTION OF PIERHEAD.
(CAISSONS)
LONGITUDINAL SECTION ON CAISSON, AFTER BEING SUNK.
CROSS-SECTION OF BREAKWATER.
(BLOCKS)
Mattress
Concrete
Mine-tube
H.W.
O.
L.W.
Basalt facing
Blocks
Parapet
SIDE ELEVATION OF BREAKWATER.
PLAN OF CAISSONS.
3rd caisson
2d caisson
1st caisson
Cylinder with mine-tube
Temporary wood deck
PLAN OF BLOCKS.
Scale of Meters

Fig. 18.

breakwater, Fig. 18, is formed by a monolithic mass of concrete, moulded on the spot between two side walls of basalt masonry. Before moulding the superstructure, the interstices between the blocks and the holes for the passing of the bars were filled as well as they could be with concrete.

Caissons.—When giving its consent to the plan for the Scheveningen Harbour, the War Department imposed the condition that the pier-heads should be constructed in such a manner that they could be destroyed by dynamite in time of war, and, therefore, that six mine tubes should be placed in each pier-head, 7.50 m. (24 ft. 7 in.) apart, from center to center. The tubes were to extend to a depth of about —2.50 m. A. P., and to have a profile of 0.75 by 0.90 m. ($2\frac{1}{2}$ by 3 ft.); they were to be made of cast iron and to be closed up water-tight in the concrete mass of the pier-head.

As it was impossible to fulfil these conditions if the pier-heads were to be constructed of inclined blocks, the writer had to look for another solution, and he proposed to place, in each pier-head, three large steel caissons filled with concrete, each caisson containing two mine tubes, which system was finally adopted.

As the caissons were to be placed at an average depth of —4 m. A. P., and, when placed, should emerge sufficiently above the high-water level to be pumped out and filled with concrete, they had to have a height of 6.35 m. (20 ft. 10 in.). Their length was fixed at 15 m. (49 ft. 3 in.), and their width varied from 6 to 7.70 m. (19 ft. 8 in. to 25 ft. 3 in), according to the increasing width of the pier toward the end. They were made of steel plates, 6 mm. ($\frac{1}{4}$ in.) thick, strengthened with corners, the plates of the upper part, which would emerge at low water after the sinking of the caisson, being fastened together with screw-bolts, as they were to be taken off after the concrete filling was sufficiently strong.

As it was thought advisable not to pour in the concrete filling under water, the caisson, Fig. 18, was divided into several compartments, by placing in it two cylinders, fixed at the bottom, and united to the sides by cross-plates, the caisson being thus divided into ten compartments, each of which was provided with an orifice to let in the water, when the caisson was ready to be sunk.

The caissons were built from this design at a dockyard near Amsterdam, and had to be towed to Ymuiden and from there over

FIG. 1.—MATTRESS FOR SCHEVENINGEN HARBOUR.

FIG. 2.—CAISSON FOR SCHEVENINGEN HARBOUR.

FIG. 3.—CAISSON, AFTER BEING SUNK.

sea to Scheveningen, the distance between this place and Ymuiden being 46 km. (28½ miles). A draft of 2.50 m. (8 ft. 2½ in.) was given to each caisson by ballasting it with a layer of concrete on the bottom, about 0.50 m. (1 ft. 8 in.) thick (Fig. 18). Before the caissons were placed, the sea bottom, which previously had been covered with mattresses, was carefully leveled with rubble and gravel, a diver assisting at this work. As all this work was completed, and the weather seemed favourable for the work, the caisson, which had been kept ready in the North Sea Canal at Ymuiden, was brought from there by a tugboat, which took about five or six hours; the caisson was brought as near as possible to the coast by the tugboat, after which it had to be towed into its place by hawsers, passing over windlasses (Fig. 2, Plate VI). The caisson having arrived at its place, all the sluices of the orifices, except two, were opened, and the corresponding compartments filled with water until the caisson had reached the bottom and settled, which was generally done in 10 min. (Fig. 3, Plate VI). Immediately after a caisson had been sunk it was filled with concrete, supplied by a mixer placed on the dunes, and brought by railway over the completed part of the breakwater.

First, the compartments which had remained dry at the sinking were filled, two other compartments, in the meantime, being pumped empty by a hand pump, were filled next, and so on until the eight outer compartments were filled. This, on an average, could be done in two days, working day and night.

After the two cylinders of the caisson were emptied, the mine tubes were placed in them, and the remaining space was filled with concrete.

The system adopted has proved satisfactory, although the transportation, sinking and filling of the caissons has been difficult work, which could be done only during very fine weather, it sometimes happening that all preparations for the placing of a caisson were made in vain, and the work had to be postponed, on account of a sudden change of the wind and weather.

Once it happened that a caisson, brought from Ymuiden, was thrown off its anchors by an unexpected rising of the waves at the moment of being sunk, and was shipwrecked on the neighbouring coast where it was destroyed by a gale the next day. At another

time, when the concrete filling of a caisson had just been completed, a gale came up suddenly, destroying some of the steel plates and making some serious breaches in the concrete. These breaches were afterward filled with concrete in bags below the high-water level, and with brick masonry above that level, the whole being left finally in very good condition.

The steel cross-plates in the caisson, which make it possible to fill it without depositing the concrete under water, prevent the concrete of the different compartments from sticking together, so that no monolithic mass can be obtained in this way. Yet the concrete in each compartment represents a monolith of about 90 000 kg. (88½ tons) which seems sufficient for this kind of work. Moreover, the concrete in the different compartments of the end caisson has been fastened together by strong iron bars, which pass through the cross-plates.

The opening in the breakwater, between the end of the inclined blocks and the first caisson, has been filled, below low-water level, with concrete in bags, and above that level with specially moulded concrete blocks placed by a small derrick crane.

On the caissons, the upper sides of which extend to about 1 m. above high-water level, the superstructure of the pier-heads has been formed, like that on the blocks, by a monolithic mass of concrete, moulded between side walls of basalt masonry, the upper ends of the mine tubes being enclosed in the concrete and locked by an iron cover (Fig. 18).

Protecting Toe.—As soon as possible after the placing of the blocks and caissons, a protecting toe was formed on the outer sea face, by depositing a rubble mound of basalt, the stones weighing about 500 kg. each. A smaller toe was deposited on the inner, or harbour face, to serve during the execution of the work.

Concrete.—The concrete for the blocks and caissons was composed of 5 parts of gravel from the Rhine, 3 parts of sand, 1 part of Portland cement and ½ part of trass (trass being a puzzolana which is found near Andernach on the Rhine and which, when mixed with Portland cement in mortar and concrete, enables them to resist the corroding action of sea water).

Cost.—The cost of the breakwaters can be estimated at 800 000 guilders ($320 000), that is, as the total length is 2 by 320 m. (700 yd.), at 1 250 guilders per m. ($457 per yd.).

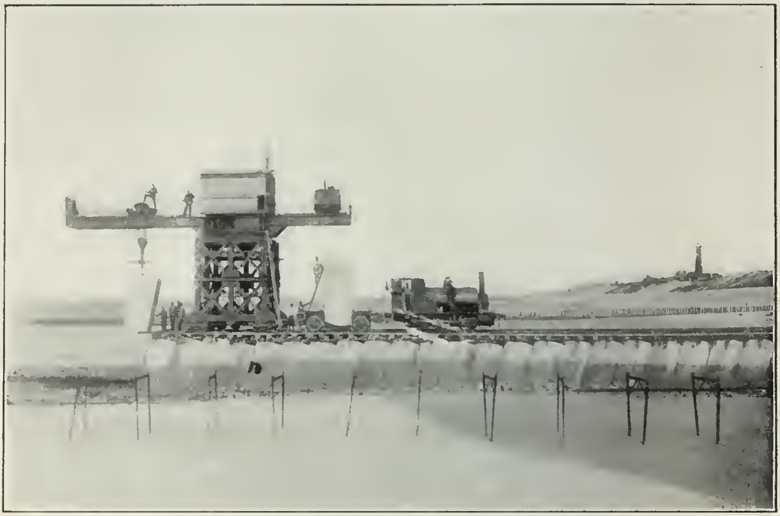


FIG. 1.—TITAN, SCHEVENINGEN HARBOUR.

FIG. 2.—BREAKWATER, SCHEVENINGEN HARBOUR.

Dredging.—The breakwaters, Fig. 2, Plate VII, are to be completed in 1904. The channel in the outer harbour has yet to be dredged, which will be done by a suction dredge of small draft, the sand being carried out to sea in fine weather, or piled upon the shore north of the harbour by a tube which can pass through the north breakwater at three places where holes have been made for that purpose.

The quantity of sand to be dredged is estimated to be 170 000 cu. m. (182 cu. yd.).

Harbour Extension at Amsterdam and Rotterdam.

During the past ten years the two great seaports of Holland, Amsterdam and Rotterdam, have extended their harbours considerably, to correspond with the development of their traffic and the increasing dimensions of steamships visiting North European ports. The Harbour of Amsterdam, which is 24 km. (15 miles) from the North Sea, and is connected with it by the North Sea Canal with its Harbour of Ymuiden, contains several basins and docks.

Since 1894 three new basins for sea-going vessels have been formed, with a total area of 35 hectares (86½ acres), and existing basins have been deepened. The quay length, affording to sea-going vessels a depth of from 8.50 to 9.50 m. (28 to 31 ft.) below the ordinary water level, has been increased from 4 900 m. (5 355 yd.) in 1894, to 6 875 m. (7 515 yd.) in 1904. The City of Amsterdam has expended nearly 9 000 000 guilders ($36 000 000) on the extension and reconstruction of its harbours.

The North Sea Canal is in course of reconstruction in order to make it navigable for the largest vessels which can pass the main lock at Ymuiden.

The width on the bottom is to be increased from 36 to 50 m. (164 ft.), and the depth from 8.5 to 9.80 m. (32 ft. 2 in.) below ordinary water level. New swing-bridges with a free width of 55 m. (180 ft.), have been built over the canal.

The Harbour of Rotterdam, on the New Maas (a branch of the Rhine), at a distance of 33 km. (20½ miles) from the sea, is connected with it by the New Maas and the New Waterway, the latter having its outlet at Hoek van Holland between two piers. The depth of the channel has been increased, since 1894, from 6.5 m.

(21 ft. 4 in.) to 7.5 m. (24 ft. 8 in.) below ordinary low water, the tidal range being, on the average, 1.70 m. (5 ft. 7 in.) at Hoek van Holland, and 1.39 m. (4 ft. 7 in.) at Rotterdam.

The depth between the piers at Hoek van Holland is now 10.40 m. (34 ft. 2 in.) at ordinary high water, the largest vessels being able to enter the river mouth. To maintain the depth between the piers at Hoek van Holland and in the sea in front of the mouth, about 600 000 cu. m. (780 000 cu. yd.) have to be dredged yearly, at a cost of 120 000 guilders ($48 000).

The area of the basins in Rotterdam, accessible to sea-going vessels, has been increased from 108 hectares (267 acres) in 1894 to 183 hectares (452 acres) in 1904, two large basins, the Maashaven, with an area of 58 hectares (143 acres), and a depth of 8.50 m. (28 ft.) at ordinary low water, and the Schiehaven, with an area of 7.32 hectares (18 acres) and a depth of 8 m. (26 ft. 3 in.) at ordinary low water, and several smaller basins, having been constructed. The length of quay walls, affording a depth of 5 to 8.5 m. (16½ to 28 ft.) to sea-going vessels at ordinary low water, has been increased, since 1894, by 3 670 m. (4 000 yd.).

The City of Rotterdam has spent since 1894 about 8 800 000 guilders ($3 500 000) on the extension and reconstruction of its harbour works.

TRANSACTIONS

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HARBORS.

MARITIME PORTS OF FRANCE.

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Translated from the French by
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The coasts of France have a total extent of 2 710 km. (1 684 miles); of which 1 223 km. (760 miles) are on the North Sea and the English Channel, from the Belgian frontier to Audierne; 862 km. (536 miles) on the Atlantic Ocean, from Audierne to the Bidassoa River; and the remainder, 625 km. (388 miles), on the Mediterranean.

The ports on these coasts number more than 560, of which at least 500 serve exclusively, or almost so, the coastwise trade and the fishing industry. Port facilities, established and maintained at the expense of the State, are to be found in nearly 200 of these ports, their number having increased only slightly since 1892.

In France, the State, represented by the Minister of Public Works, constructs and maintains the ports, lighthouses and bea-

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cons. It does not usually take part either in the establishment or operation of railroads or other port facilities.

The cost of improving the ports is not entirely defrayed by the State. The industries, chambers of commerce, cities and departments frequently participate in the expense by subsidies, sometimes reimbursable but which, more often, are gifts.

The establishment and operation of railway tracks placed on the quays are almost always in the hands of the railroad companies whose systems terminate at the respective ports.

The public plant, placed at the disposal of all, is usually established by the chamber of commerce but sometimes by the city.

Conditions of Access to Ports.

Notable modifications, especially in regard to access, have been made in the conditions surrounding the establishment of facilities in ports.

The east jetties of Dunkerque and of Calais have been reconstructed, with the result that the width of entrance has been brought from 70 to 120 m. (230 to 394 ft.) at Dunkerque, and from 100 to 135 m. (328 to 443 ft.) at Calais. The east jetty at Fécamp has been extended. The jetties at the fairway of the Adour have been made full.

The channels at Dunkerque, Calais and Boulogne have been deepened, so that the bottom is now from 3.5 to 4 m. (11.5 to 13 ft.) below the level of the lowest tides.

At the jetty heads of Calais, a depth of 5 m. (16.4 ft.) below that level is maintained. In the outer port of Boulogne, they have excavated to a depth of 9.75 m. (32 ft.) below the zero of the marine charts,* one basin being 700 by 320 m. (2 300 by 1 060 ft.).

Similar deepenings of 0.50 and 1 m. have been realized in the outer channels at Dieppe, and Hâvre, respectively. At Cette the construction of the Dellon spur jetty has reduced the wave action in the passes and permits the realization of a definite increase of depth in the east pass, the one most used by large vessels.

The bars which exist across the mouths of the Loire (Barre des Charpentiers) and the Adour have been lowered, that of the former to a depth of 2.2 m. (8.3 ft.) by cutting a channel 200 m.

* In France the zero of the marine charts is generally found slightly below the level of the lowest tides (equinoctial spring tides).

(756 ft.) wide on the bottom, and that of the latter to a depth of at least 2 m. (6.56 ft.).

Important works of dredging and bank protection have been executed in the Seine, the Loire, the Charente and the Garonne to improve the access to the ports of Rouen, Nantes, Rochefort and Bordeaux.

In the Seine the dikes have been partially consolidated or reconstructed and have been extended, the north dike as far as the meridian of Saint-Sauveur, and the south dike as far as Honfleur. Reefs which existed in the river bed have been cut down about 1 m., and this has greatly ameliorated the difficulties of navigation in the upper river while the greatest depth to be found in the estuaries outside the jetties is slightly above the zero of the marine charts. Ships drawing 5.6 m. (18.4 ft.) can ascend to Rouen in ordinary slack water and those drawing 7 m. (23 ft.), during the highest tides.

In the Loire they have succeeded, with great difficulty in certain places, in maintaining the bed of the river at an elevation of 0.30 m., the floatage at high tide not exceeding 3.80 m. (12.46 ft.) in neap tides, or 4.80 m. (15.7 ft.) in spring tides. A lateral canal, 15 km. (9.36 miles) long, between Carnet and Martinière, was opened to navigation in 1894, and repeated dredging in the stretches below and above it, now permits the passage of vessels drawing 6 m. (19.7 ft.) as far as Nantes at any stage of the tide.

In the River Charente the draft of vessels destined for Rochefort has been limited by the Fouras Reef, which is covered with 5.4 m. (17.71 ft.) of water at ordinary high tide, or 7 m. (23 ft.) at spring tide. The dredging, now being done and which is already greatly advanced, assures an incease of draft of at least 2 m. (6.6 ft.), thus permitting ships drawing 8 m. (26 ft.) to pass up at nearly all tides and those drawing 9 m. (29.5 ft.) at spring tides.

In the Garonne, works which have been carried on since 1892 by a new method have resulted in lowering the reefs, especially those of Bassens and of Callion, which have been reduced about 0.5 m. At present the draft at all high points is between 3.5 and 4 m. (11.5 and 13 ft.) at low tide. Bordeaux, therefore, receives ships drawing from 7 to 8 m. (23 to 26 ft.), according to the state of the tide.

This channel deepening is being carried on in other places as the needs of navigation develop. In a comparatively short time the channel at Dunkerque will be deepened 1 m. more, while the channel at Hâvre and the pass through the Barre des Charpentiers will be deepened 1.5 m. more.

Interior Arrangement of the Ports.

The interior arrangements of the ports also have been subjected to great modifications since 1892. Breakwaters and landing stages have been established or improved in a large number of fishing ports. Quays and wharves have been constructed at Dunkerque, Calais, Fécamp, Hâvre (the Tancarville Canal), Rouen (the lumber and petroleum docks), Caen, Cherbourg, St. Malo and St. Servan, Le Légué St. Brieuc, Nantes, Royan, Bordeaux and Bayonne.

At Dieppe the tidal quays have undergone great development, so as to allow the establishment of a regular steamboat service for England at fixed hours, and to accommodate the fisheries.

At Paullac, on the Gironde, and on the Adour, private companies have provided large wharves. The Paullac wharf or pier is 375.25 m. long by 23.83 m. wide (1 231 by 78 ft.). Vessels can lie on both sides. The depth of water alongside is at least 8.5 m. (27.64 ft.) at low tide.

Open docks are being constructed at Boulogne and Marseilles. Floating docks have been established at Tréport, Hâvre (the petroleum docks), Paimpol and Pontrieux. The east dock at Calais used by the petroleum interests is in process of transformation; it is to be supplied with a new entrance lock which will soon be completed.

In order to permit larger vessels to have access to the docks it is sometimes sufficient to lower the bottom by dredging and rock removal, as has been done at Hâvre (at the Eure Docks), Rouen, Cherbourg, St. Malo, St. Nazaire, Nantes, Cette and Bastia (old port).

At Caen, increase of draft has been obtained by raising the level of the water in the canal, connecting with the sea, 0.5 m.

In cases where these processes do not permit the increase of depth desired, new locks have been constructed, as at Dunkerque and Ouistreham, or new gates provided, as at Hâvre and St. Nazaire.

The Dunkerque and Ouistreham locks have been opened to navigation; the new gates at Hâvre and St. Nazaire are not yet finished.

The new gate chambers at Hâvre will have the sill 4.5 m. (14.76 ft.) below extreme low tide, which will be 1.65 m. (5.4 ft.) lower than that of the Transatlantic Lock; this will allow most vessels to be docked at any time, whereas at present their entrance is limited to not more than three hours of each tide. The same conditions will prevail at St. Nazaire, where the bottom of the new lock will be at 0.6 m., or 2.70 m. (8.86 ft.) below the present one.

Such extensive deepenings, either in the entrance channels of the ports or in the maritime rivers, can only be maintained by vast dredging operations, which amount annually to 1 000 000 cu. m. at Dunkerque, 320 000 cu. m. at Calais, 530 000 cu. m. at Boulogne, 270 000 cu. m. at Hâvre, 200 000 cu. m. in the Seine, 460 000 cu. m. at St. Nazaire, 1 500 000 cu. m. in the Loire, 625 000 cu. m. in the Garonne and 650 000 cu. m. at the Adour Bar. These quantities have increased very considerably since 1892, when the dredging for maintenance amounted to only 500 000 cu. m. at Dunkerque, 350 000 cu. m. at Boulogne, 180 000 cu. m. at Hâvre, 300 000 cu. m. at St. Nazaire and about 250 000 cu. m. in the Loire; at that time no dredging was done on the Adour Bar.

The number of graving and repair docks has only increased slightly since 1892; new dry docks, however, have been constructed at Dieppe, Bordeaux and Bayonne and a ship railway, on the Labat system, at Nantes. In addition, two of the dry docks, at Hâvre and St. Nazaire, as well as the old ship incline at Bordeaux, have been lengthened.

Table 3 gives the comparative channel depths in 1892 and 1902, respectively, for the more important ports, and the depths over sills of the principal entrance locks at different stages of the tide.

PLANT.

The outfit of machinery and other apparatus for port use which in 1892 had already attained a great development, has continued to be improved ever since. The increase applies chiefly to the public facilities established by the chambers of commerce, the private plants remaining about as they were.

TABLE 3.—DEPTHS IN FRENCH PORTS.

PORTS.	Year.	CHANNEL. Elevations of bottom plane.	CHANNEL. Depth of water at: Ebb tide.	CHANNEL. Depth of water at: Flood tide, neap.	CHANNEL. Depth of water at: Flood tide, spring.	LOCK ENTRANCES TO BASINS. Widths.	LOCK ENTRANCES TO BASINS. Available lengths of lock chambers	LOCK ENTRANCES TO BASINS. Elevations of sills.	LOCK ENTRANCES TO BASINS. Depth of water on sills. Ebb tide.	LOCK ENTRANCES TO BASINS. Depth of water on sills. Flood tide, neap.	LOCK ENTRANCES TO BASINS. Depth of water on sills. Flood tide, spring.	Water areas.	Length of quays.	Wharfage areas.	Remarks.
		Meters.	Meters.	Meters.	Meters.	Meters.	Meters.	Meters.	Meters.	Meters.	Meters.	Hectares.	Meters.	Hectares.	
Dunkerque..	1892	—2.60	4.30	7.50	8.50	21.00	147.00	—1.55	3.25	6.45	7.45	50.00	7.934	54.70	
" .	1903	—4.00	5.70	8.90	9.90	25.00	170.00	—5.00	6.70	9.90	10.90	50.00	8.409	55.85	
Calais........	1892	—2.50	4.40	8.20	9.50	21.00	119.68	—1.78	3.68	7.48	8.78	31.50	6.725	24.83	
"	1903	—3.50	5.40	9.20	10.50	21.00	119.68	—1.78	3.68	7.48	8.78	31.50	6.725	24.83	
Boulogne...	1892	—2.00	4.64	9.06	10.90	21.00	100.00	—0.14	2.78	7.20	9.04	27.83	2.933	3.69	
"	1903	—3.75	6.39	10.81	12.65	21.00	100.00	—0.14	2.78	7.20	9.04	27.27	2.933	4.25	
Dieppe......	1892	—2.50	4.87	9.62	11.67	18.00	150.00	0.00	2.37	7.12	9.17	24.25	3.898	18.01	
"	1903	—3.00	5.37	10.12	12.17	18.00	150.00	0.00	2.37	7.12	9.17	23.90	3.940	18.80	
Hâvre.......	1892	—2.00	4.65	8.15	9.85	30.50	......	—2.85	....	9.00	10.70	74.18	12.265	43.49	
"	1903	—3.00	5.65	9.15	10.85	30.50	......	—2.85	....	9.00	10.70	77.38	13.421	48.47	
Rouen.......	1892	.....	......	......	......	......	......	......	....	....	....	37.70[1]	3.767	40.00	[1] Water areas, in hectares: 1892. 1903. Lumber docks.... 4.70 5.25 Petroleum docks. 7.00 12 00
"	1903	......	......	......	......	......	......	......	....	....	....	48.25[1]	5.286	60.00	
St. Servan. St. Malo...	1892	+3.50	1.00	5.65	8.85	18.00	71.50	3.57	0.96	5.58	7.75	48.20	3.235	16.90	
St. Servan. St. Malo...	1903	+3.50	1.00	5.65	8.85	18.00	71.50	3.57	0.96	5.58	7.75	48.20	4.480	19.82	
St. Nazaire..	1892	—3.30	4.93	7.23	8.73	25.00	......	—3.30	....	7.23	8.73	33.00	4.300	25.15	
" ..	1903	—5.50	7.13	9.43	10.93	25.00	......	—3.30	....	7.23	8.73	33.00	4.300	25.15	
Nantes......	1892	......	......	......	......	......	......	......	....	....	....	.....	4.200	5.30	
"	1903	......	......	......	......	......	......	......	....	....	....	.....	5.000	9.00	
La Pallice...	1892	—5.00	6.95	9.66	10.80	22.00	165.00	—4.00	5.95	8.66	9.80	28.00	2.130	40.00	
" ...	1903	—5.00	6.95	9.66	10.80	22.00	165.00	—4.00	5.95	8.66	9.80	28.00	2.130	40.00	
Rochelle.....	1892	—0.60	1.55	5.26	6.40	16.50	......	0.93	....	5.59	6.73	7.72	2.044	1.06	
"	1903	—0.60	1.55	5.26	6.40	16.50	......	0.93	....	5.59	6.73	7.72	2.044	1.06	
Bordeaux...	1892	......	.....	......	......	22.00	152.00	—3.00	3.97	6.93	7.50	.....[2]	2.700	71.00	[2] Water area of floating docks: 10 hectares.
" ...	1903	......	......	......	......	22.00	152.00	—3.00	3.97	6.93	7.50	.....[2]	4.665	79.00	
Bayonne.....	1892	—3.00	3.45	4.80	5.85	22.00	......	......	....	....	....	.....	2.410	3.90	
"	1903	—5.00	5.45	6.80	7.85	22.00	......	......	....	....	....	.....	2.710	5.50	
Cette........	1892	......	......	......	......	......	......	......	....	...	....	47.00	6.187	12.31	
"	1903	......	......	......	......	......	......	......	....	....	....	47.00	6.187	12.31	
Marseilles....	1892	......	......	......	......	......	......	......	....	....	....	172.06	18.118	71.36	
" ...	1903	......	......	......	......	......	......	......	....	....	....	191.99	21.6.0	95.50	
Nice.........	1892	......	......	......	......	......	......	......	....	....	....	6.80	1.100	1.25	
"	1903	......	......	......	......	......	......	......	....	....	....	6.80	1.100	1.60	

Since 1892 sheds for protecting the merchandise on the quays have been constructed at Dunkerque, Calais, Boulogne, Hâvre, Nantes, La Pallice, Bordeaux and Marseilles.

The number of hoisting machines has been notably augmented. The use of floating derricks has become somewhat less general. Steam cranes have been established at Dunkerque, Hâvre, Nantes, La Pallice, Bordeaux and Bayonne; hydraulic cranes at Dunkerque, Calais, Rouen, Hâvre, Bordeaux and Marseilles. Electric cranes are at length coming into extensive use. They have been installed at Calais, Boulogne and Hâvre, and are now being set up at Nantes. High-powered hoists have been introduced at Hâvre (120 000 kg.) (132 tons), Bordeaux (80 000 kg.), Nantes (60 000 kg.), Calais (40 000 kg.), Dieppe and Bayonne (30 000 kg.).

Electric lighting to facilitate vessel movements and work by night has received new applications at Dunkerque, Calais, Dieppe, Rouen and Bordeaux.

The length of tracks, superficial areas of quay sheds and the number of lifts belonging to the chambers of commerce are given in Table 4 for the principal ports in 1892 and 1903, respectively.

Methods of Execution.

The methods of executing work in connection with the ports have changed very little since 1892. But the power of mechanical agencies has increased and made common certain methods which formerly were only used under special and rare circumstances.

Compressed air is now frequently used in establishing works which formerly would have been founded by pumping at great expense and under the greatest difficulties. It serves equally for subaqueous construction, and, under better and more economical conditions, for quays which formerly would have been constructed either by superposed blocks or by immersed concrete. Such is the case for the walls of the Pinède Basin at Marseilles.

Compressed air permits of laying foundations at greater depths and of giving vertical facings to jetties and dikes instead of resting them on rip-rap with a talus at the foot, dangerous for ships. The new east jetties at Dunkerque and Calais (upon half its length), and the heads of the dikes forming the new entrance to the port of Hâvre, have been established thus with great benefit to navigation.

TABLE 4.

Ports.	Length of Railroad Tracks.		Area Covered by Sheds.		Hoisting Apparatus, Number of.		Remarks.
	1892	1903	1892	1903	1892	1903	
	Meters.	Meters.	Meters.	Meters.			
Dunkerque..	32 200	40 949	20 500	29 400	1	31[1]	[1] 2 floating derricks of 40 000 and 10 000 kg.; 29 cranes from 1 500 to 3 000 kg.
Calais	16 428	21 428	22 400	25 840	4	26[2]	[2] 1 shears of 40 000 kg.; 1 crane of 10 000 kg.; 2 of 5 000 kg.; 2 of 3 000 and 2 000 kg.; 20 from 1 500 to 750 kg.
Boulogne....	5 412	7 240	3 800	6 004	5	5[3]	[3] 5 cranes of 40 000, 15 000, 10 000, 4 500 and 3 000 kg.
Dieppe.......	9 161	9 249	2 820	2 820	19	19[4]	[4] 3 cranes of 5 000; 5 of 3 000; 11 of 1 500 kg.
Hâvre........	37 271	39 570	61 483	94 966	28	69[5]	[5] 1 shears of 120 000 kg.; 8 floating derricks from 10 000 to 1 250 kg.; 2 cranes of 3 000 kg.; 30 of 1 500 kg.; 28 of 1 250 kg.
Rouen.......	27 340	30 438	12 000	12 000	21	34[6]	[6] 2 cranes of 30 000 and 1 000 kg.; 2 of 2 500 kg.; 30 of 1 250 kg.
St. Malo... / St. Servan.	10 850	10 850	....	....	2	2[7]	[7] 2 cranes of 20 000 and 1 000 kg.
St. Nazaire..	10 000	11 600	....	....	....	....	No public facilities conceded by Chamber of Commerce.
Nantes	6 037	9 549	....	4 157	14	31[8]	[8] Shears of 60 000 kg.; 1 crane of 15 000 kg.; 3 of 3 000 to 5 000 kg.; 26 of 1 500 kg.
La Pallice...	9 364	11 477	....	9 364	....	17[9]	[9] 1 crane of 10 000 kg.; 16 of 1 500 kg.
Rochelle.....	2 700	2 700	....	....	....	....	No public facilities conceded by Chamber of Commerce.
Bordeaux....	17 100	36 000	10 900	15 700	22	74[10]	[10] 1 shears of 80 000 kg.; 1 crane of 10 000 kg.; 20 of 3 000 and 2 000 kg.; 52 of 1 500 kg.
Bayonne.....	1 000	1 470	....	....	4	10[11]	[11] 1 shears of 30 000 kg.; 2 cranes of 3 000 and 5 000 kg.; 7 of 1 500 kg.
Cette........	1 825	1 825	....	....	....	1[12]	[12] 1 crane of 15 000 kg.
Marseilles...	42 400	44 400	45 722	56 711	31	54[13]	[13] 1 shears of 120 000 kg.; 3 cranes of 25 000, 8 000 and 4 000 kg.; 3 of 3 000 kg.; 10 of 3 000 and 1 000 kg.; 37 of 1 250 to 1 000 kg.
Nice..........	....	920	....	....	1	1[14]	[14] 1 crane of 10 000 kg.

Note.—1 000 kg. approximates closely to 1 ton of 2 240 lb.

The increase of mechanical power permits of constructing works with blocks of much greater dimensions, and of securing better conditions of stability. Thus the lower courses of the jetties at Bizerte and Heyst are formed of blocks 9 m. (29.5 ft.) high, 25 to 30 m. (82 to 98 ft.) long and weighing not less than 3 000 or 4 000 metric tons, whereas, only recently, the weight of the largest blocks ever laid did not exceed 100 or 120 tons. The caissons of metal in which these enormous blocks were made, as well as those which enclosed the quay piers at Tunis, were put in place by flotation.

The ever-increasing development of dredging operations has led to the increase of the power of the dredging machines. The plant at Dunkerque, for example, which in 1892 comprised three suction dredges of a total capacity of 510 h. p. and one dipper dredge of 90 h. p., is now composed of four suction dredges of combined 1 310 h. p. and two dipper dredges developing together 310 h. p. The same is correspondingly true at nearly all the ports where dredging is carried on.

During recent years there have been put in service many dipper dredges having engines of 400 to 650 h. p. raising from 300 to 400 cu. m. (392 to 523 cu. yd.) per hour (Boulogne, Hâvre, the Seine, the Charente), and suction dredges of 400 to 600 h. p., raising from 300 to 600 cu. m. per hour. Dredging prices, accordingly, have been much lowered, and they have now fallen, first cost of plant and amortization not included, for dipper dredges to 0.54 franc per cu. m. (8.3 cents per cu. yd.) at Calais or Boulogne, and even to 0.4 franc per cu. m. (6.2 cents per cu. yd.) in the Loire, with short disposal distance; while, for suction dredges, the figures reached have been 0.18 franc per cu. m. (Boulogne, Barre des Charpentiers), and even 0.15 franc (Adour Bar) (2.7 cents and 2 cents per cu. yd., respectively).

The conveyance of dredged material by forcing it through pipes has been improved and developed. At Bordeaux, where this process is used on a large scale, the dredged material is spread on the river banks at distances up to 2 500 and 3 000 m. (1.53 and 1.86 miles). An installation has just been completed which will increase the distance to 4 000 m. (2.5 miles in round numbers) by the use of a relay pump operated by an electric motor.

The utilization of electricity has improved the workshop organi-

zations and developed the use of mechanical appliances by permitting the substitution of a single central machine to produce energy in place of a number of steam engines scattered all over the works in proximity to the various shaping machines, hoisting apparatus, pumps, air compressors, mortar mixers, shop tools, etc. The shop organization has thus been simplified while becoming more elastic, more perfect and more economical.

Steel-concrete construction has begun to be used, notably in the construction of pier wharves at Cherbourg and Nantes, and in the completion of the jetties of the Sables d'Olonne; but, thus far, in marine works this new material has not been considered of much importance, as the fissures which often develop, especially in foundation work, are of a nature to raise doubts as to its durability, because of the decomposition of mortars by sea water.

Expenses.

In 1903, the maintenance of the staff attached to the service of the maritime ports (engineers, sub-engineers, superintendents, clerks, tenders, and other agents of inferior rank), both for maintenance and for new works, cost about 4 250 000 francs ($850 000). This annual sum has not varied appreciably since 1892.

In 1903 the expenses of maintenance of the works, including gross repairs, were 9 040 000 francs ($1 808 000); in 1892 they were 7 240 000 francs ($1 408 000).

The improvement of the ports, from 1892 to 1903 inclusive, has cost altogether 231 142 959 francs ($46 228 592), as follows:

Treasury funds	138 047 341	francs
Subsidies from industries	89 113 673	"
Advances by industries	3 981 945	"
Total	231 142 959	francs

In addition, the State reimbursed loans, previously made by the industries, amounting to 66 354 172 francs.

Table 5 shows, for the period from 1892 to 1903, inclusive, the appropriations made for the improvement of the principal ports and for increasing the public facilities dependent upon the chambers of commerce.

TABLE 5.—Cost of Port Improvements.

Ports.	Treasury funds.	Subsidies by commercial interests.	Total.	Subsidies.	Cost of plant.
	Francs.	Francs.	Francs.	Per centage.	Francs.
Dunkerque	12 539 259	9 900 000	22 439 259	41.1	7 413 383
Calais	7 145 177	1 278 186	8 423 663	15.2	169 645
Boulogne	4 234 979	2 410 000	6 644 979	36.4	120 232
Dieppe	3 553 372	2 743 868	6 297 240	43.6	....
Hâvre	18 901 826	10 108 666	29 010 492	34.2	2 704 987
Rouen	21 734 711	8 570 586	30 305 297	28.3	975 221
St. Malo / St. Servan	406 509	1 731 299	2 137 808	80.9	88 956
St. Nazaire	6 686 232	7 471 478	14 157 710	52.8	....
Nantes	12 429 324	3 873 000	16 302 324	23.7	1 371 014
La Pallice	2 594 014	....	2 594 014	....	1 101 836
Rochelle	542 740	....	542 740	....	....
Bordeaux	13 458 559	11 840 000	25 298 559	46.8	1 812 000
Bayonne	3 458 000	3 089 000	6 547 600	47.2	561 500
Cette	4 255 019	....	4 255 019	....	130 840
Marseilles	13 536 418	3 920 000	17 456 418	22.5	?
Nice	382 885	542 000	924 885	58.6	1 154

TABLE 6.—Port Commerce.

Ports.	1892.			1902.		
	Vessels.		Merchandise.	Vessels.		Merchandise.
	Numbers.	Tonnage.	Tons.	Numbers.	Tonnage.	Tons.
Dunkerque	5 913	2 910 758	2 438 781	4 920	3 455 629	2 716 580
Calais	4 904	1 239 924	354 318	4 468	1 647 973	488 220
Boulogne	5 225	1 524 579	463 252	5 332	3 423 304	665 492
Dieppe	3 327	920 320	531 573	3 513	839 249	497 411
Hâvre	12 505	5 678 135	2 881 835	12 422	6 273 480	3 175 698
Rouen	5 361	1 982 073	1 788 219	4 762	2 161 967	2 321 564
St. Malo	3 258	471 953	....	3 324	520 892	382 765
St. Servan	1 654	62 258	....	856	68 449	65 228
St. Nazaire	2 180	1 723 610	1 227 668	4 690	1 821 122	1 187 517
Nantes	2 803	407 770	385 035	5 686	1 191 279	1 086 713
La Pallice	1 877	158 818	60 291	1 659	986 072	290 381
Rochelle	7 621	584 637	361 838	6 539	537 150	426 205
Bordeaux	15 633	3 525 007	2 400 620	21 929	4 081 945	2 691 618
Bayonne	1 660	514 006	631 666	1 392	543 550	681 839
Cette	4 619	2 269 044	738 958	3 569	2 110 211	822 252
Marseilles	16 877	9 791 485	4 576 992	17 008	13 233 274	6 488 067
Nice	1 812	364 340	165 070	2 376	595 782	223 074
Other ports	91 240	8 036 197	5 093 593	117 417	12 853 150	6 323 258
Total for France	189 469	42 264 941	24 102 649	224 861	56 316 688	30 836 916

Traffic.

Table 6 gives, for the years 1892 and 1902 (the last year for which the official statistics of the customs have been published), the commercial business (imports and exports combined) done at the principal ports, and for the whole of France.

The duties received by the State reached the following figures:

	1892.		1902.	
Import duties........	430 778 221	francs	381 136 169	francs
Statistical duties.....	7 053 515	"	7 539 109	"
Navigation duties....	8 283 271	"	7 510 306	"
Accessories..........	5 101 946	"	5 592 645	"
Total..........	451 216 953	"	401 778 229	"

TRANSACTIONS
AMERICAN SOCIETY OF CIVIL ENGINEERS.

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1904.

Paper No. 10.

HARBORS.

THE PREPARATION AND USE OF CONCRETE BLOCKS FOR HARBOUR WORKS.

By I. Hiroi.*

The durability of cement mortar and concrete in sea water has been a subject of serious controversy between cement manufacturers and scientists during the past fifteen years, owing to frequent failures of sea works in which concrete blocks have been used. Dr. Michaelis, one of the highest authorities on the subject, has gone so far as to declare, on the ground of his careful researches, that Portland cement as now manufactured is incapable of resisting indefinitely the action of sea water, and, as an improvement, he has proposed the addition of trass, pozzuolana and siliceous materials of similar nature, capable of taking hold of the calcium hydrate liberated in cement during the progress of induration. A similar view is held by MM. Le Chatelier, Ferret and a host of chemists and engineers. Indeed, the theory seems to be an established one, as far as the present knowledge of the chemistry of cement and certain laboratory experiments are concerned. The experiment car-

* Civil Engineer, Tokyo, Japan.

ried out at the Isle of Sylt under the auspices of the Prussian Government in 1898* has established the superiority in strength of trass-cement mortar over ordinary cement mortar when used in sea water, but it neither proved the destructibility of the latter nor the indestructibility of the former. It must ever be kept in mind that the phenomena—both physical and chemical—in the sea cannot be reproduced exactly in laboratories on land, and, consequently, the results of experiments in the latter should always be estimated accordingly.

Of more than 12 000 concrete blocks of all sizes and of different compositions, used by the writer in harbor works during the past ten years, there has not been the slightest indication of failure. On the contrary, the protection given by coatings of animal, vegetable and mineral origin to the surfaces of the blocks, has rendered them almost impregnable against the incessant action of the sea. The protection thus afforded by Nature, which would render a comparatively weak artificial stone, when properly made, as lasting as the natural stone used as its ingredient, is unknown in laboratory experiments.

Careful examinations of blocks placed in sea water for more than ten years at Hakodate and Otaru, Japan, and a few other localities have disclosed no signs of disintegration, but have found them harder and stronger than when they were put in.

With regard to the use of siliceous materials, the writer's experiments have shown that trass and good pozzuolana may be used with advantage, whenever they are easily procurable. The most important factor in insuring the durability of a block is, however, the mode of fabrication, most of the failures of concrete blocks being traceable to the lack of proper attention to some of the important details.

The object of the paper is to discuss briefly the essential points to be observed in the preparation of concrete blocks for use in sea works, presenting, as a subject for further discussion, a short description of the method followed by the writer, at present, at the Otaru Harbor Works.

The subject will be discussed under the following headings:

* Bericht ü. d. Verhalten hydraulischer Bindemittel im Seewasser.

The qualities and proportions of ingredients,
Mode of fabrication,
Treatment of blocks.

The Qualities and Proportions of Ingredients.—Of the ingredients used in making a block, cement is evidently the most important. The four essential qualities of cement for use in making blocks are

1.—Fineness, the coarse grains being nothing more than so much sand, which, when ground, still retains the power of induration long after setting.

2.—Least amount of alumina, since the formation of sulpho-aluminate of lime is the principal cause of decomposition of cement in sea water.

3.—Slowness of setting, as considerable time generally transpires between mixing the concrete and filling the moulds.

4.—Constancy of volume in setting and requisite strength within specified time—for obvious reasons.

The following specifications are being enforced on the cement used at the Otaru Harbor Works:

"The cement shall be so fine that residue in a sieve of 900 meshes per sq. cm. will not exceed 10 per cent. of the whole.

"The quantity of alumina contained in the cement shall not be more than 8 per cent.

"The insoluble matter in the cement shall not exceed 3 per cent.

"The cement shall not commence setting within one hour of mixing with water.

"The cement in setting shall not show any change in shape or volume.

"Briquettes made of 1 part of cement with 3 parts of standard sand, and immersed in sea water, shall have a tensile strength of not less than 9 kg. per sq. cm. after 1 week, gradually increasing to 12 kg. per sq. cm. in 4 weeks, the amount of that increase to be not less than 2 kg. per sq. cm."

The modes of testing are also strictly prescribed, the main feature being the use of sea water as outside medium for physical tests.

A series of experiments with siliceous materials of different kinds has shown that Rhenish trass is almost unique in its action,

decidedly improving the quality of cement when used in sea water; but with pozzuolanas and tuffs found in Japan, the results have been various according to their qualities, and the conclusion reached, with regard to their use, is that good pozzuolana or tuff may be used with advantage, in amount, however, not exceeding that of the cement. For determining the quality of pozzuolana or tuff, physical tests have been found to be the only reliable means, chemical analyses merely serving to discriminate between the inferior cements of the same group.

At Otaru, the tuff extracted from the bluff close to the blockyard has been largely used since 1901. The following is its average chemical composition:

	Percentage.
Silica	66.900
Alumina	14.517
Ferric oxide	5.600
Lime	3.783
Magnesia	1.993
Alkali	6.825
Loss by ignition	0.382

TABLE 7.—TENSILE STRENGTH, IN KILOGRAMMES PER SQUARE CENTIMETER.

Length of time immersed in sea water.	Cement Mortar.		Tuff-Cement Mortar.	
	1 cement, 2 sand.	1 cement, 3 sand.	1 cement, ½ tuff, 3 sand.	1 cement, 1 tuff, 4 sand.
1 week	15.82	8.68	8.96	4.90
4 weeks	19.78	11.76	13.62	12.16
2 months	21.65	14.56	20.44	16.26
4 "	23.90	15.55	23.13	22.53
6 "	24.56	16.87	29.83	24.95
1 year	26.87	23.36	33.77	29.88
y ears	31.09	19.73	33.80	30.99

The proportions are by weight. The cement is from the Asano Cement Works.

The results of tests given in Table 7 show the comparative strength of tuff-cement and ordinary mortars, from which it will

be seen that the strength of the former, although much lower than that of the latter at first and for some time, becomes greater in about four months and steadily increases.

It is evident that each of the ingredients ought to influence, by its nature and amount, the quality of the concrete. Sand for mortar should be hard and coarse. That the size of grains has influence on the strength of mortar is shown by the results of tests given in Table 8, whence it will be seen that sand with a mixture of all sizes of grains gives the best results, so far as the tensile strength of the mortar is concerned.

TABLE 8.—TENSILE STRENGTH, IN KILOGRAMMES PER SQUARE CENTIMETER, OF BRIQUETTES MADE OF 1 CEMENT AND 3 SAND, BY WEIGHT.

Kind of sand.	Strength for 1 week.	Strength for 4 weeks.
Grains 2 mm. and under, not sifted.	13.15	17.04
Sifted in sieves with 60 and 220 meshes per sq. cm.	9.72	11.81
Grains under ½ mm., not sifted	8.57	10.75

The washing of the sand can generally be dispensed with unless the quantity of mud is excessive.

For ballast, a mixture of hard broken stone and gravel is perhaps most suitable, because, when broken stone only is used, the mass resists packing, while gravel alone makes the mass too yielding for ramming.

Concerning the proportions of the ingredients, the most important matter is the ratio of cement to sand, an excess of one or the other producing deleterious results. Guided by the results of well-known experiments at La Rochelle and a series of tests made at Hakodate, the writer in all his work adhered to a proportion of 1 part of cement to 2 parts of sand. The superiority of this proportion has been proved by several tests. The results of one series are given in Table 9.

In all other tests the proportion of 1 cement to 3 sand failed to give the blocks the strength calculated as necessary to with-

stand with safety the pressures and shocks, either of transport or of wave action at the expiration of the periods specified.

The proportions of mortar and ballast have been determined in such a way that the former will always be slightly in excess of the voids in the latter. The proportions of ingredients had been, on the average, 1 cement, 2 sand, 2 gravel and 2 broken stone, before tuff was introduced. Since then, the following combination, by volume, has been used:

1.0 cement, 0.8 tuff, 3.2 sand, and 6.4 ballast;

the equivalent ratio by weight of cement to tuff being 1 cement to ½ tuff.

TABLE 9.—TENSILE STRENGTH OF BRIQUETTES KEPT 24 HOURS IN AIR AND THEN IMMERSED IN SEA WATER.

Time elapsed.	Tensile Strength, in Kilogrammes per Square Centimeter.		
	Neat cement.	1 cement, 2 sand.	1 cement, 3 sand.
1 week	43.52	21.56	14.33
1 month	53.56	24.83	17.02
3 months	59.72	26.25	20.11
6 "	61.05	30.18	23.51
1 year	32.22	30.26	23.85
2 years	13.88	35.03	25.32
3 "	10.55	40.24	28.48
4 "	8.88	46.13	30.40
5 "	6.81	47.93	30.22
6 "	5.93	48.76	34.84

The cement is from the Alsen Cement Works.

The only drawback to the use of tuff, especially when in greater proportions, is the slowness with which the blocks harden, requiring them to be kept in the yard a somewhat longer time.

The kind of water used in mixing has but little influence on the strength of concrete; fresh water, wherever easily procurable, is, however, without doubt, to be preferred to sea water. The quantity

of water to be used for mixing depends on the nature of the ingredients and the humidity of the air. At Otaru it has been, on the average, 13.5% of the volume of concrete. The effect of varying the quantity of water is shown by the results of tests given in Table 10. The smaller the quantity of water used, within certain limits, the better is the result in the early period of growth in strength.

TABLE 10.—TENSILE STRENGTH OF BRIQUETTES MADE OF 1 CEMENT AND 3 BEACH SAND, IN KILOGRAMMES PER SQUARE CENTIMETER.

Quantity of water, by weight.	1 week.	4 weeks.	6 months.	1 year.	2 years.	4 years.
One-seventh.......	6.67	11.26	22.07	18.99	21.49	25.36
One-ninth..........	8.05	14.24	23.14	24.84	24.40	25.20
One-eleventh......	11.52	14.77	21.18	22.77	24.31	25.32

Mode of Fabrication.—It is hardly necessary to say that thorough mixing of the ingredients is the first requisite in the operation of block making. For this purpose machine mixing is to be preferred to hand mixing. At the Otaru Harbor Works the cement and tuff are mixed in a rotating cylinder and stored in that form until used, when they are mixed with sand in the same mixer. For mixing with ballast a continuous mixer of Carey-Latham type is used. This mixer is shown on Plate VIII.

The most important quality of a block is its density. The strength, weight and impermeability of the blocks depend entirely on the density of the concrete. This important quality can be secured only by thorough ramming, although a tolerably dense block could be produced by using an excess of water in mixing, making mortar or concrete of semi-liquid consistency, which would readily fill a mould when poured. Such concrete is, however, decidedly inferior in strength to rammed concrete, as will be seen from the results of tests given in Table 11.

For ramming, it has been customary to use a number of hand rams weighing from 6 to 12 kg. At the Otaru Harbor Works, a pneumatic ram (Plate VIII) has been constructed and put to work with success. The weight of the ram is 225 kg., the stroke, 15 cm., and the number of strokes 250 per min. The ram is suspended from the overhanging arm of the gantry which carries the boiler, engine and compressor. By this means not only a large saving of labor is effected, but an element of uncertainty attending the results of all onerous manual work is, to a great extent, removed.

TABLE 11.—Crushing Strength of Concrete, in Kilogrammes per Square Centimeter.

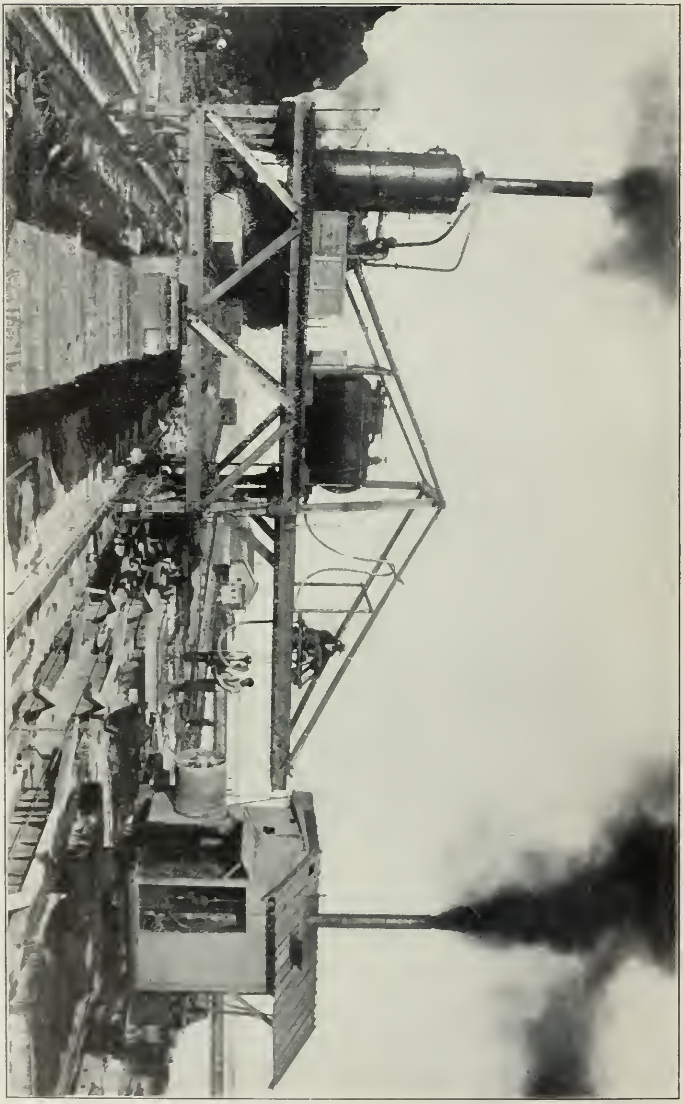
	2 months.	6 months.	2 years.
Rammed (13% water)	185.1	226.2	258.0
Not rammed (18% water)	78.3	110.8	112.5

The proportion is, 1 cement, 2 sand and 4 tuff, and the block is a 10-cm. cube.

Ramming is performed best in layers of about 15 cm. in thickness. Since the inroad of sea water between the layers is as dangerous as that into the concrete itself, a most complete union of successive layers should be effected. For this purpose each layer should be scratched all over with pointed tools before another layer is laid; this is one of the most important precautions in the fabrication of rammed concrete blocks for use in sea water. The mould, after being filled, should be covered with wet mats or cloth to prevent rapid drying. There is no doubt that the longer the block is kept in the mould the better the result will be, but as this would require the provision of an almost indefinite number of moulds the time of removal is generally limited to from 3 to 5 days after filling.

Treatment of Blocks.—As to the treatment of blocks after removal from the moulds, the results of tests given in Table 6 have been instructive.

From which it appears that the best result could be obtained by keeping blocks in fresh water before immersing in the sea. But



Concrete Mixer, and Block-Making Plant, Otaru Harbour Works.

as this is seldom practicable, it is advisable, as a second-best course, to keep the blocks covered and to water them frequently until they are sufficiently hardened to bear transport, when they may be put in the sea, as there is no advantage in keeping them on land unless kept continually wet.

TABLE 12.—TENSILE STRENGTH OF BRIQUETTES MADE OF 1 CEMENT AND 2 BEACH SAND, KILOGRAMMES PER SQUARE CENTIMETER.

	2 months.	4 months.	9 months.	1 year.	2 years.
Briquettes kept 24 hours covered in air and immersed in sea water	19.89	23.30	31.25	31.41	27.79
Kept 2 months in fresh water and then placed in sea water.	21.76	25.91	32.52	33.94	34.60
Kept 2 weeks in air and then immersed in sea water	24.73	27.30	30.48	31.96	29.72
Covered and watered for 2 weeks in air and immersed in sea water	23.91	24.51	31.47	34.38	33.45
Exposed in air and watered from time to time for 2 months and placed in sea water	24.51	26.81	33.34	31.03	30.87

Conclusion.—In this paper two qualities of concrete, strength and durability, have been considered together. As far as the chemical action of sea water is concerned, the concrete could be durable without being particularly strong; for, as already stated, the durability of concrete placed in the sea depends solely on the protective covering of organic and inorganic nature and the colloidal deposit, which, having once closed all the pores, forever excludes the sea water from the mass, leaving it intact, and almost as though kept in fresh water. Apart from the good qualities of the ingredients, without which no sound concrete could ever be made, the most important point to be observed in making the concrete durable enough to receive the necessary coating from Nature, is to make it dense and massive. With regard to the proportions of the ingredients, the question, although still unsettled, is not so much one of durability, where large mass is concerned, as of economy, and the requisite strength generally determines the limit.

It is but little more than 70 years since hydraulic cement began to be used in sea works, and sufficient time has not yet elapsed either to prove or disprove the various claims made for it. Meanwhile, its faultless behavior up to the present time, wherever it has been used properly, speaks for its value, and with it engineers may hope, by due attention to all the details described, to produce materials which will outlast those of Nature.

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Paper No. 11.

HARBORS.

CONCRETE BLOCKS AT OSAKA HARBOUR WORKS, JAPAN.

By S. Shima.*

General Description.

The new Harbour of Osaka is on the west shore of the city at the outlet of the Aji River. The works consist of an outer harbour protected by two jetties converging toward the entrance, and an inner basin sheltered by a breakwater which starts from the north bank of the River Kizu and is connected to the south jetty. Of the total area, 2 640 acres, thus enclosed, the portion along the shore, comprising an area of 121.8 acres, is reclaimed land; and two wet docks are to be constructed therein. At the extremity of the new land, facing the entrance of the harbour, a steel landing pier 1 500 ft. long and 90 ft. wide is already constructed. The remaining part of the enclosed area is to be dredged to a certain depth—especially the portion between the entrance and the pier, to a depth of 28 ft. below low water to permit vessels to have access to the pier.

The work was undertaken by the Municipality of Osaka, com-

* Civil Engineer, Osaka, Japan.

mencing in October, 1897, and is now near completion. The total estimated cost amounts to 22 570 400 yen.

Cement concrete blocks, in the making of which the writer was appointed as engineer in charge, and whereof he has now the honour to present a brief report, were thrown at random to cover the surface of the rubble-base jetties. All the blocks were of uniform size, 6 ft. long by 5 ft. wide and 4 ft. deep, weighing 8 tons. Up to date, 54 355 such blocks have been made, and they are deemed sufficient to complete the jetties. The block-making was started in November, 1899, and ended in May, 1904. During this period the writer has investigated the question of how reliable blocks can be made at a moderate cost, and has obtained the results shown in the following pages.

Arrangement of Block Yard.

As before stated, nearly 60 000 blocks were required for the construction of the jetties, and this number had to be completed within five years. Now, assuming one-sixth of a year to be lost, owing to bad weather or other accidents, it was necessary to manufacture 12 000 blocks in 300 days, or 40 blocks per day. The block yard (Plate IX) was so arranged as to answer this requirement. It was established at the starting point of the breakwater on the right bank, at the outlet of the River Kizu. It was flat ground, specially reclaimed for the purpose, covering an area of 21 acres. The block-seasoning ground occupied about one-half of the whole area, as the blocks were to be seasoned for three months without being moved from their original positions. Along the eastern border of the ground a road for block-trucks was excavated to a depth of 3 ft. below ground level, so that the tops of the trucks were level with the ground. At the northern extremity of the road a high wooden trestle pier was built from which a block-loading machine could load the blocks into lighters.

On the opposite side of the seasoning ground a mill-stage, three stories high, was constructed (Plates IX and X), on the second and first stories of which five mortar mills and ten concrete mixers, respectively, were set. Elevators were provided on both sides of the stage, one serving to raise gravel and two others, cement and sand.

PLATE IX. VOL. LIV. PART A.
TRANS. AM. SOC. CIV. ENGRS.
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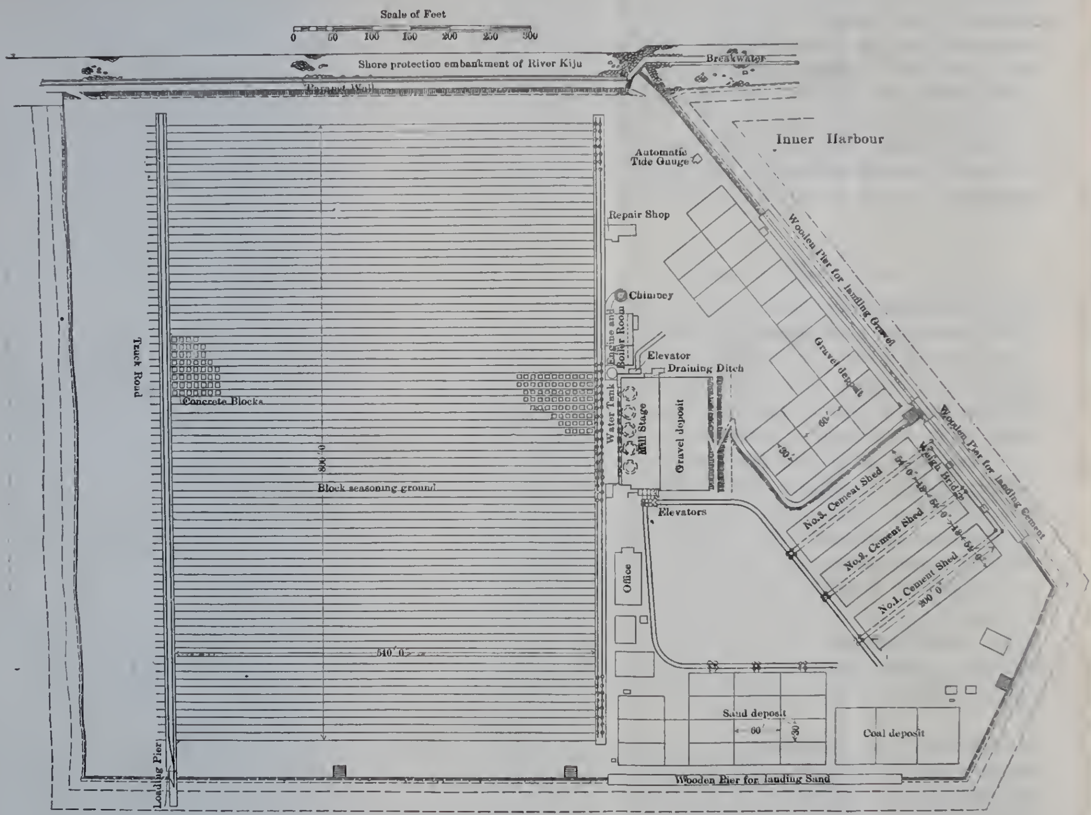
GENERAL ARRANGEMENT OF BLOCK YARD.



Near the stage an engine-house (Plates IX and X) was built for an engine of the horizontal surface-condensing compound type, of 120 h.p. Three cement sheds of 1 188 sq. yd. each., a sand deposit of 3 200 sq. yd., and a gravel deposit of 4 600 sq. yd. were prepared along the north and west quays of the yard. In front of these sheds, wooden stages were erected, and 1½-ton cranes traveled along the quay to unload the raw materials.

All parts of the yard were connected, either by heavy rails, or by Decauville rails, for facility of transportation.

Ingredients of Concrete.

The ingredients of the concrete were Portland cement, sand and gravel. The cement was limited to that furnished by two native firms, the Asano Cement Company and the Onoda Cement Company. To supervise the manufacture of the cement, two inspectors were stationed permanently with each firm, and only cement which was sealed by them was allowed to be delivered. When the cement arrived at the block yard, an inspector examined the weight and dampness, and samples were extracted from 2 to 5% of the barrels of every cargo. The samples were submitted to both rigid chemical and mechanical tests. If any of these samples did not fulfil the conditions of the specification, the cargo was rejected. Extracts from the principal clauses of the specification are as follows:

"Chemical Analysis: If a sample of the cement shows, by chemical analysis, that it contains either more than 1% of anhydrous sulphuric acid ($S\ O_3$), or a trace of calcium sulphide ($Ca\ S$), or more than 3% of magnesia ($Mg\ O$), or more than 4% of ferric oxide ($Fe_2\ O_3$), or that the hydraulic index is less than 42, the cement shall be rejected.

"Tests of Setting: Cements which begin to set in less than 1 hour or finish setting in less than 3 hours or later than 12 hours shall be rejected.

"Tests of the Tensile Strength: The tensile strength of the neat cement briquettes after 7 days shall be not less than 285 lb. per sq. in., and after 28 days, not less than 500 lb. per sq. in.; that of the standard sand mortar briquettes after 7 days shall be not less than 110 lb. per sq. in., and after 28 days, not less than 215 lb. per sq. in. (the standard sand mortar consists of 1 part of cement to 3 parts of standard sand)."

The cement cost 23 yen per ton. The sand was obtained at the mouth of the River Yamato, about 1½ miles south of the block yard. Its grains were clean, hard, and angular. Such valuable qualities in the sand, in addition to its facile transportation, were really very favourable to the work. The sand was screened before being used, on a sieve of $\frac{3}{16}$-in. square meshes. It cost 0.44 yen per cu. yd.

With regard to the gravel, no special variety was specified, but it was limited to that from sea beaches. It was obtained mostly from the northwestern coast of Osaka Bay. The grains were hard and clean, but not very sharp. They were also screened between 2-in. and $\frac{3}{8}$-in. sieves. The gravel cost 1.06 yen per cu. yd.

Prior to the commencement of the work, it was a question whether the quantity of gravel which could be obtained from the vicinity would meet the demand. Consequently, three Blake stone-breakers were reserved. Fortunately, it was found that ample gravel could be obtained at a comparatively cheap cost, the breakers became useless, and no broken stone was used throughout the work.

Mixing of Mortar and Concrete.

The proportions of the concrete were as follows:

Cement............25 lb. to 1 cu. ft. of sand;
Sand.............. 2 } by volume.
Gravel............ 3 }

Since each block contained 120 cu. ft., the corresponding ingredients were as follows:

Cement....................1 500 lb.;
Sand...................... 60 cu. ft.;
Gravel.................... 90 cu. ft.

These ingredients were all measured beforehand in their respective stores, and every 600 lb. of cement, 12 cu. ft. of sand, and 18 cu. ft. of gravel were carried off on Decauville wagons.

The mortar mills, details of which are given in Plate XII, consisted of annular trenches, 15 ft. in diameter, in which a rake and three runners or rollers revolved at a rate of 7 rev. per min. to churn the mixture. Thus, each mill was capable of turning out 24 cu. ft. of mortar in 18 minutes.

PLATE X. VOL. LIV. PART A.
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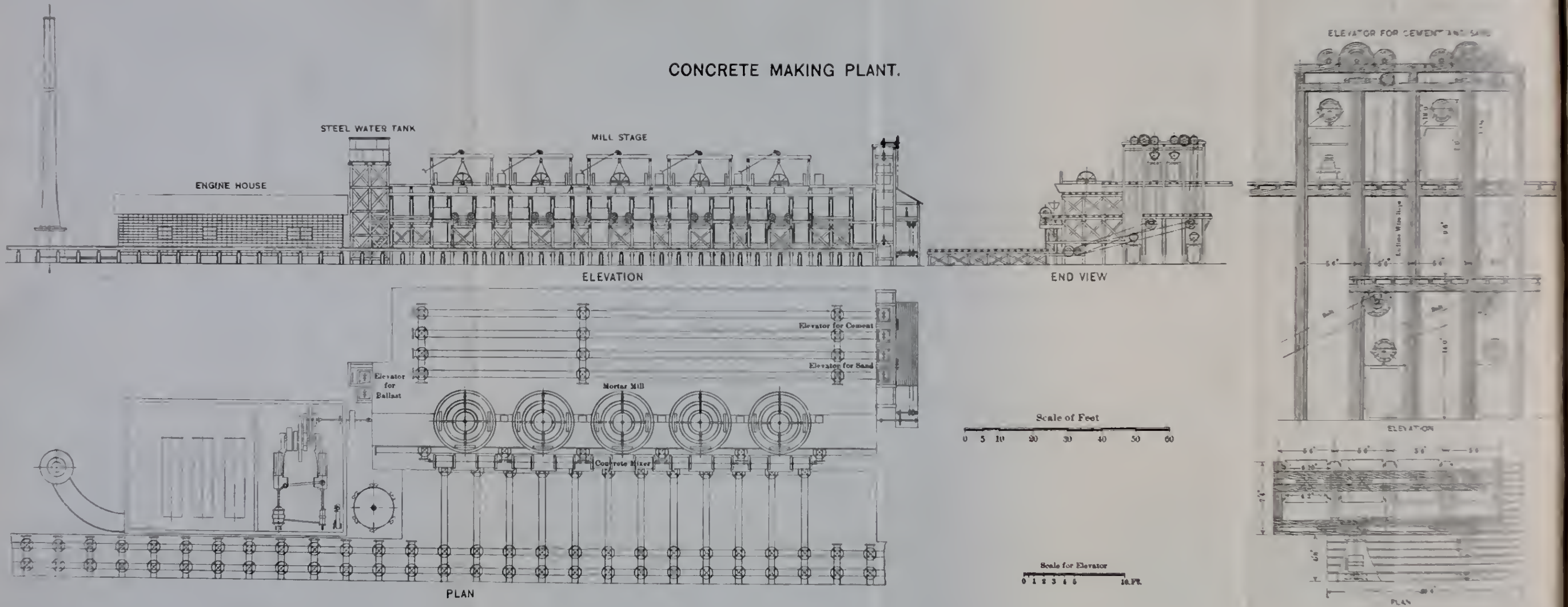
CONCRETE MAKING PLANT.



The concrete mixers (Plate XIII) had the form of hollow cylindrical drums, 5 ft. long by 3½ ft. in diameter, having eighteen radiated stays inside, which served to mix the contents when the drums rotated on their horizontal axes. Each mixer was able to make 24 cu. ft. of the matrix in 5 min., at a rate of 8 rev. per min.

The mills and the mixers were manufactured specially in accordance with the detailed drawings and specifications, in a native workshop. They cost 2 150 and 530 yen each, respectively.

The process of making mortar and concrete with this machinery was as follows:

In the first place, a wagon load of cement (600 lb.) and two wagon loads of sand (24 cu. ft.) are tipped into the mortar mill and churned together for 10 minutes. Then, a cock of a water-tank standing nearby filled with sea water is opened; the water in the tank is sprinkled over the mixture through numerous small holes made at intervals in the iron pipe running around the upper edge of the trench. The quantity of water varied every day according to the temperature, the humidity of the atmosphere and the nature of the ingredients. In summer, the water used for gauging was from 8 to 9.5% of the weight of the whole mixture, and in winter, from 7 to 8 per cent. The gauging continues for 8 minutes; the sluice provided in front of the mill is opened, and, at the same time, a wooden piece hanging from the top of the rake is dropped so as to block the bottom of the sluice. The wooden piece pushes the mortar out and stacks it in front of the rake, and at every arrival of the rake at the sluice the mortar is pushed out of the mill so long as the revolution continues. A few revolutions are sufficient to clear the mill; subsequently the motion is stopped, and the sluice is shut. As soon as the revolution of the rake and runners commences, new raw materials are charged, mixed, and gauged as before, and so on.

This mortar is discharged into two wagons on the first story, each receiving one-half, or 12 cu. ft. These loads are fed into different mixers, and turned over together with the gravel which has been poured in previously. The gravel is washed with water before being charged into the mixers in order to remove the sand and soil which might still adhere to the grains, and, at the same time, to facilitate the adherence of the mortar to them. The mixing is completed in

5 minutes, and the concrete so formed is discharged into the wagons on the floor of the stage, and is carried to the block-seasoning ground, and there cast into moulds ready to receive it.

The mills and the mixers were worked, as before stated, on an intermittent system, and as is inevitable in this system, there was more or less inconvenience, due to the frequent handling which was required to produce and to stop the motion. But, on the other hand, the regularity of charging the raw materials and the perfect incorporation of the mortar, which were admirably secured by these machines, made up fully for this defect. (See "Miscellaneous Experiments.")

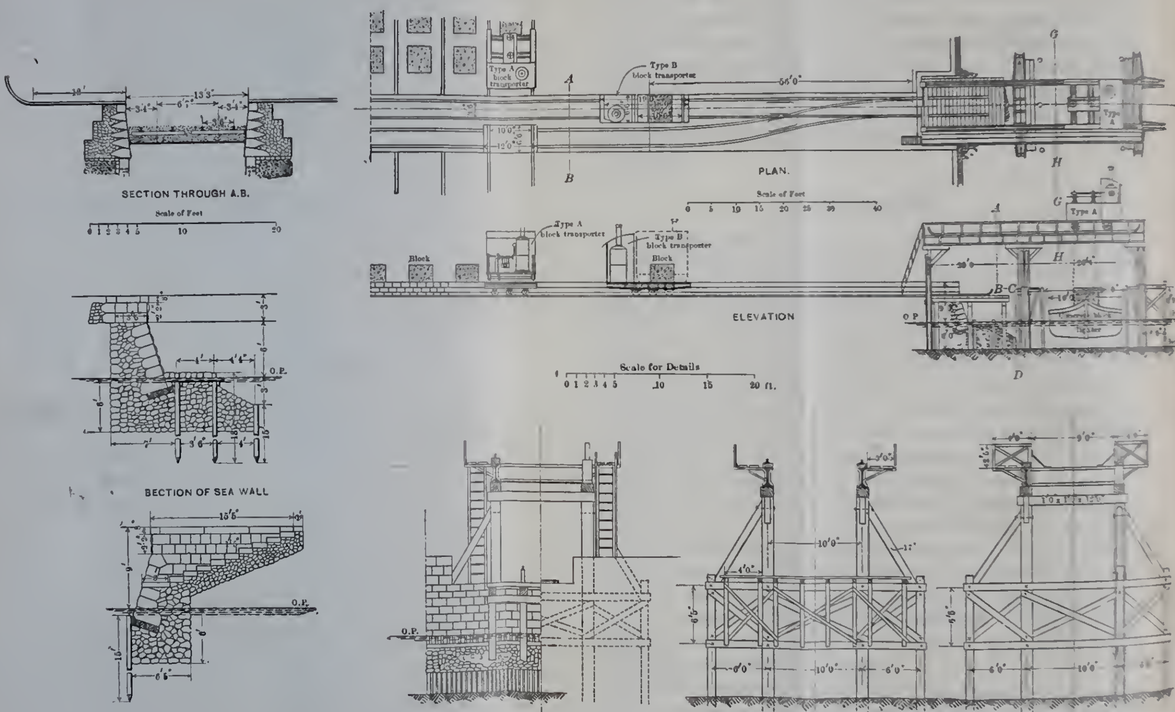
Making of Blocks.

The wagons used to carry the concrete were of the Decauville type, each capable of holding 12 cu. ft. They ran along light railways, temporarily laid between and over the moulds disposed in adjacent tiers (Plate XIV). The moulds and the planks were covered previously with petroleum to prevent the adherence of the concrete to them. The corners and possible leakages were made water-tight with clay and a certain kind of bark. First, a wagon load of concrete was tipped into the mould; two coolies forced the concrete with shovels into the corners and sides; six coolies rammed forcibly and uniformly over the surface of the concrete with three iron-shod rammers, each weighing 28 lb. and having a base 6 in. square; another coolie rammed along the sides with a battledore, weighing 10 lb. (Plate XV). After 9 minutes the ramming was finished. The water oozing out on the surface was removed with sponges, and before charging the next load the surface was scraped with rakes to a depth of about 2½ in., so as to secure perfect adherence of the layers of concrete. Each mould was completed in 1½ hours, by charging ten wagons or 120 cu. ft. of concrete. The labour of the block-making cost 0.62 yen per block.

The blocks were covered with mats for two days after their completion, and during that period they were kept damp by being sprinkled occasionally with water so that they might set properly, and three months later they were transported to their destination.

It would not be out of place to give a description of the age of

GENERAL ARRANGEMENT FOR TRANSPORTATION AND SHIPMENT OF CONCRETE BLOCKS.



SECTION THROUGH A.B.

SECTION OF SEA WALL

PLAN.

ELEVATION

SECTION THROUGH A.B.C.D.

SECTION THROUGH G.H.

SECTION THROUGH G.H

the moulds and planks. They were made of "Matsu," a species of pine tree. At the beginning of the work 740 moulds were prepared, and after two years 300 more were added to supply the losses due to damage and rot. During this period 15 839 blocks were made, so that, on the average, the moulds were used 20 times. With regard to the planks, the damage was more rapid; probably direct contact with the ground and the moisture hastened their decay, so that after 2½ years there was scarcely a plank left fit for use. The life of the moulds and planks would not have been so long if used in making blocks for more particular work, since the use of partly deformed moulds and planks would have made the blocks unfit for such work.

Transportation of Blocks.

The transportation of the blocks was effected, at first, by a lifting-machine and block-trucks (Plate XIV), and afterward by a loading-machine (Plate XI). The lifting-machine consisted of a strongly braced truck with sufficient clear space in its front to straddle over one block. Chain-slings provided with iron hooks (Plates XIV and XV) at their ends dropped from the extremities of the two arms. These arms could be raised or lowered by means of worm-gears. The loading-machine on the wooden pier previously mentioned was of the same construction as the lifting-machine, except that the chain-slings were replaced by wire ropes, which were also provided with iron hooks at their ends. The block-trucks were only platforms with space sufficient to load the blocks.

In the first place, the lifting-machine raises the block a few inches, just enough to clear the floor, and travels with it along the rails laid in the spaces between the tiers of blocks, and brings it to No. 1 Block-Truck; this truck runs on the sunken road and carries the block under the pier, while No. 2 Block-Truck comes to the starting place of No. 1 Truck and makes a way for the lifting-machine going astern over the truck road. On the other side, the loading-machine receives the block from No. 1 Truck, runs on the pier, and puts it on a block-lighter brought under the pier.

Six lighters were used for the service, each being able to carry 7 blocks. It was not uncommon to ship 80 blocks in a working day of 10 hours.

TABLE 13.—Progress in Making Blocks.

Year.	1899.	1900.	1901.	1902.	1903.	1904.
January	....	113	510	1 106	1 016	1 320
February	....	211	1 038	1 277	1 168	1 142
March	....	307	1 006	1 390	1 656	....
April	....	662	239	1 280	1 514	660
May	....	1 154	694	1 252	1 636	88
June	....	820	584	814	1 548	....
July	....	760	521	1 398	1 404	....
August	....	758	700	1 312	1 778	....
September	....	1 085	924	1 176	1 588	....
October	....	1 053	972	1 352	1 602	....
November	73	830	1 304	1 272	1 554	....
December	166	650	1 208	1 184	1 526	....
Total	239	8 403	9 700	14 813	17 990	3 210

Note.—Total number of blocks made, 54 355.

TABLE 14.—Progress in Transporting Blocks.

Year.	1900.	1901.	1902.	1903.	1904.
January	....	982	1 403	861	1 197
February	....	721	1 226	1 136	1 547
March	28	1 099	1 481	2 015	126
April	319	1 330	981	1 148	48
May	96	861	1 442	1 904	....
June	251	224	1 169	1 582	....
July	1 078	887	1 568	1 232	....
August	1 394	651	928	1 624	....
September	735	637	882	1 771	....
October	274	140	1 536	1 862	....
November	24	805	1 463	1 295	....
December	795	889	887	1 421	....
Total	4 994	9 226	14 969	17 851	2 918

Note.—Total number of blocks transported, 49 958.

PLATE

TRANS. AM.

INTER. ENG.

CONCRETE BLOCKS FOR

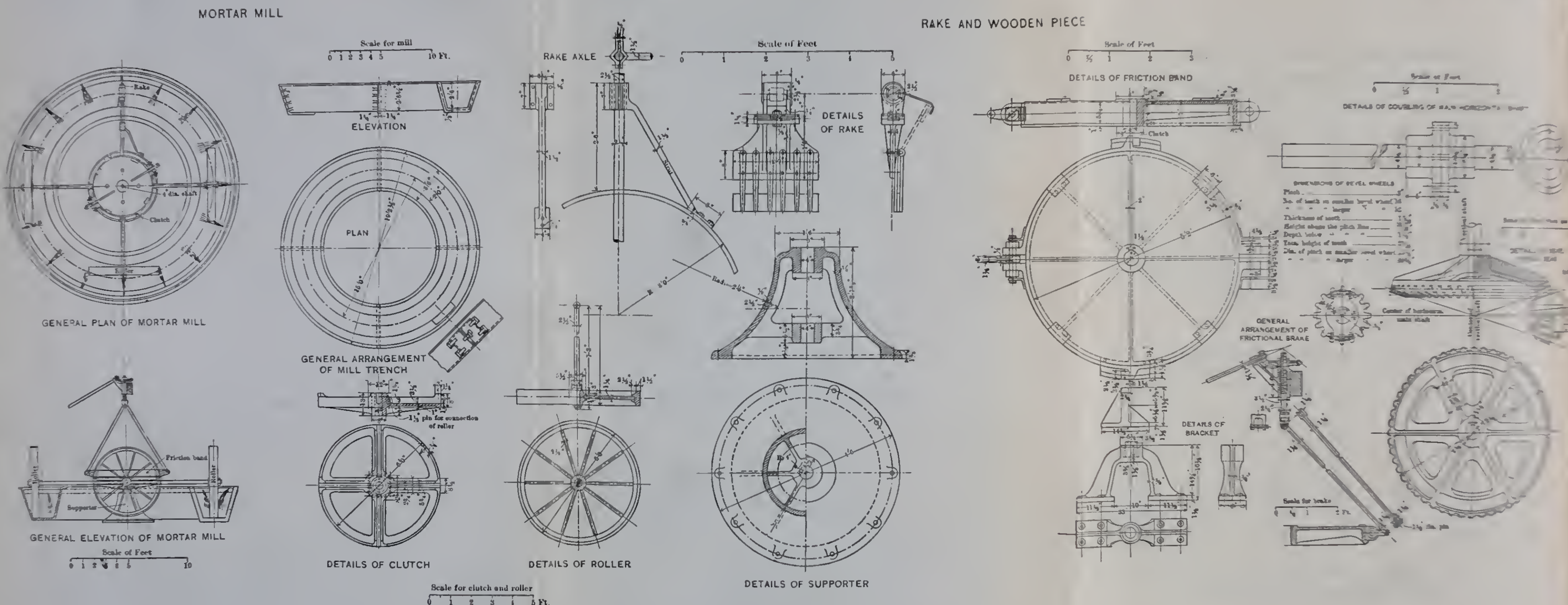


TABLE 15.—Summary of the Expenditures for Preliminary Work.

Description.	Number.	Cost, in yen.
1.5-ton portable cranes	3	8 194.070
Blake stone breakers	3	6 165.000
Engine and boilers		20 150.000
Concrete mixers	10	5 300.000
Mortar mills	5	10 750.000
Block-lifting machines	2	7 600.000
Block loading machine	1	3 900.000
Block trucks	2	7 900.000
Block moulds and planks	1 040 / 5 100	43 966.470
61-lb. rails		28 557.003
Decauville rails		9 459.370
Decauville turntables	278	8 340.000
Decauville wagons	248	15 079.500
Reclamation and quays		155 569.371
Wooden landing piers	3	16 718.770
Cement sheds and deposits		5 396.183
Mill stage and elevators		18 349.622
Truck road and loading pier		8 651.445
Engine-house	1	4 250.000
Sundry secondary works		87 243.366
Total		471 540.170

TABLE 16.—Summary of the Expenditures for Block-Making.

Description.	Amount. in yen.
Workmen	50 219.714
Coolies	154 606.793
Materials	1 153 361.882
Sundry expenditures	40 795.222
Total	1 398 983.611

Progress and Cost of Work.

The reclamation of the block yard was commenced on June 15th, 1898, and completed on January 16th, 1899. The preliminary work was carried on until November 14th, 1899, and the next day the block-making began. All the machinery and tools, except the rails and wagons, had been manufactured in Japanese workshops from special drawings. In the beginning only a few blocks were made a day. They were soon increased, however, and from April 27th, 1900, four mills and eight mixers were used, and 40 blocks were made daily; after three months this number was increased to 48, and continued so until February, 1903, when a mill and two-mixers which had hitherto been reserved, were set to work, and 60 blocks per day were made. On May 2d, 1904, the work was finished. In short, it took 4½ years to make 54 355 blocks. During this period the work was stopped entirely or partly, at times, owing to a deficiency of raw materials, bad weather, or accidents. Nevertheless, it may be said that the work has proceeded as planned, since the original plan was to make 60 000 blocks in five years.

As to the transportation of the blocks, the progress was very irregular, inasmuch as it depended upon the jetty work, which has been hampered a great deal, owing to the unfavourable conditions of the bed of the sea. Schedules of the progress, both of making and of transporting the blocks, are shown in Tables 13 and 14.

The expenditure for the preliminary work amounted to 471 540.170 yen, including the reclamation of the yard, and the blocks cost 1 398 983.611 yen, in all. Summaries referring to the foregoing expenditures are shown in Tables 15 and 16.

Miscellaneous Experiments.

The method of making and transporting blocks, which the writer has carried out at the Osaka Harbour Works, has been described in the foregoing pages. He intends, now, to add to this record a few experiments made in connection with the work. The writer's intention was, as previously stated, to make blocks of moderate cost, but which would be reliable for submarine works. Referring to the schedules given in Tables 15 and 16, the cost of the blocks can be found, and is as follows:

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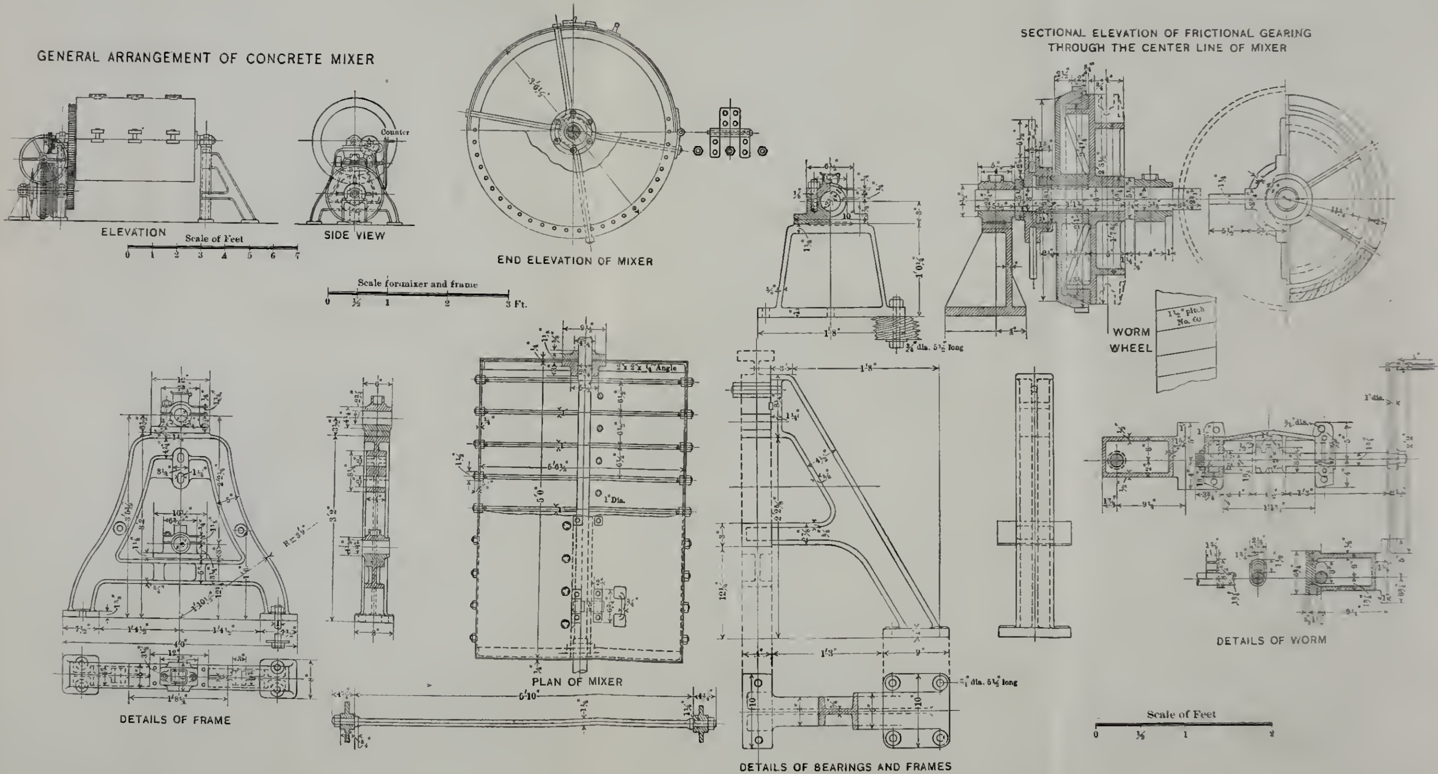
CONCRETE MIXER.



Description.		Amount, in yen.
Preliminary work		8.675
Actual work	Materials	21.219
	Labor	3.768
	Sundry expenditures	0.751
Total		34.413

Since each block weighs 8 tons, the cost of 1 ton is 4.302 yen. The writer believes, therefore, that the blocks have been manufactured at a moderate cost, in spite of the fact that the block yard was designed on a comparatively expensive scale. Now the question remains whether the blocks so made are fit for submarine works.

In the first place, turning to the proportions of the ingredients of the concrete, it was a question whether these proportions could form a concrete sufficiently impervious to resist the chemical action of sea water. For the concrete consisted of 25 lb. of cement to 1 cu. ft. of sand, and 3 parts of gravel to 2 parts of sand by volume, which makes 1:3:5, approximately by volume. Such cannot be called a very rich concrete for use in sea water; however, it must be considered that even a trifling increase in the quantity of cement would cause a remarkable increase in the total cost. As is well known, a satisfactory concrete can only be obtained with cement sufficient to fill the interstices of the sand and to encircle its grains, and the mortar so formed should encircle the gravel, provided all the ingredients be of good quality. In order to fulfil this condition, it is necessary to know the volume of the interstices of the sand and the gravel. Thereupon, the writer measured the interstices with a water-measure system, and obtained the following results:

Sand, in the same condition as was used in the work (See "Ingredients of Concrete")	35	per cent.
Sand, rammed forcibly after being charged into the measure	25	"
Gravel, in the same condition as was used in the work	36	"

The results are the average value of tests repeated ten times. The measure used for the sand was an iron box having a capacity

of 12 cu. ft., and that used for the gravel had a capacity of 18 cu. ft. In the first place, referring to the mortar, if the first case be applied, the ratio of the cement and sand becomes 1 to 2.8. But, in the practical work, the writer used three rammers, each weighing 28 lb., in order to lessen, as much as possible, the interstices of the aggregates. Hence, to determine the ratio of the mortar, the second case may be applied. Let the cement be increased by 15% of the voids; then the ratio becomes 1 to 3.5. Thus, the ratio 1 to 3 answers the purpose very well, and produces a mortar properly mixed, or, in other words, the deficiency of cement has been compensated by the ramming system.

As to the ratio of the mortar and gravel, there is no room for discussion, since the interstices of the gravel are only 36% of its volume, while the ratio of the mortar and gravel used was, practically, 2 to 3.

Next to be considered is the work of the mortar mills. As previously explained, three runners and a rake are provided in the mill. It was a question whether the runners would not deteriorate the mixture by grinding and pulverizing, thereby reducing the sharpness, coarseness, and angularity of the sand. According to the writer's personal experience, the weight of the runners (about ½ ton each) was too light to crush the grains of sand, the mixture being about 4 in. thick. On the contrary, the writer has found that the strength and incorporation of the mortar churned by the mill were thoroughly appreciable. In order to ascertain whether the incorporation was perfect or not, samples of mortar were taken from different places in two mills and treated with hydrochloric acid. The liquid was filtered, and the residue, *i. e.*, the sand, was washed, dried, and weighed. The results are as follows:

Samples taken from:	"A" mill.	"B" mill.
Inner side of the trench	71.730	72.280
" " " " "	71.150	72.593
Middle of the trench	71.467	71.933
" " " "	71.113	73.033
Outer side of the trench	71.207	71.400
" " " " "	70.933	71.666

To test the tensile strength of the mortar, the writer has made briquettes with mortar gauged in a mill, and has compared their

PLATE XIV. VOL. LIV. PART A.
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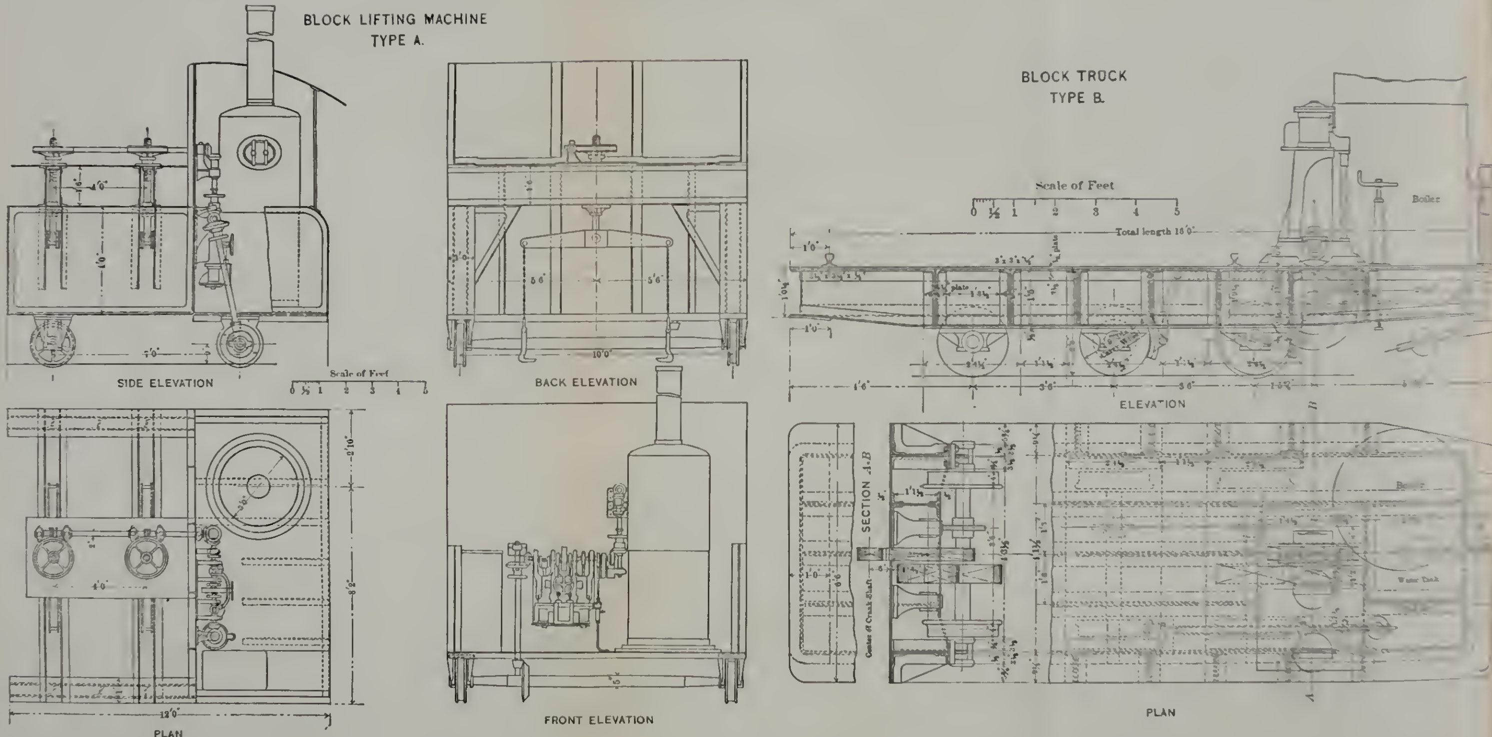
CONCRETE BLOCK TRANSPORTERS



tensile strength with that of briquettes which were made in the laboratory with the same materials, and, as nearly as possible, under the same conditions. The comparison is as follows:

Production of mortar.	Tensile Strength, in Pounds per Square Inch After:			
	7 days.	28 days.	3 months.	6 months.
Mortar mill	261.75	356.25	387.50	403.50
Laboratory	261.25	337.75	319.25	324.25

From this experiment, it will be seen that the mortar gauged in the mill is, by no means, inferior in strength to that manipulated formally in the laboratory. The writer concluded, therefore, that the mortar mixed in the mill never deteriorates in consequence of the pressure of the runners.

As to the use of sea water for gauging mortar, several opinions have been expressed by experts. The writer does not intend to quote them here. But, relying upon many successful experiments which had been previously made, to prove that concrete mixed with salt water has behaved well when used in marine works, the writer presumes that the deleterious effect of salt water on Portland cement is due mostly to the sea water in which the concrete is deposited, and not to that used for mixing, provided the water is clean and contains no dangerous elements, and that the blocks are carefully made and kept in the air for a reasonable period. He has not hesitated to use sea water to mix the concrete, and no deterioration, owing to the gauged water, has been found, to the present time.

Another question was, whether the method of making the blocks was reliable. In the first period of the work, the writer was rather disagreeably surprised to see that 264 blocks deposited in sea water had deteriorated. In making the blocks he has applied three methods: First, two wagon loads of concrete at a time were tipped into the mould and rammed for 30 minutes, then two more loads were discharged and rammed, and so on; second, observing that the first method was not satisfactory to ensure adherence between the successive layers, the concrete was scraped with rakes before loading

the next wagons; finally, the ramming was reduced to 18 minutes, the scraping being proceeded with as in the second method. Several hundred blocks were made by the first and second methods. They were seasoned for 3 months, and then deposited in sea water. After 1 to 4 months it was found that some of these blocks were lined with white precipitates, and others with slight fissures. These precipitates and fissures were all found along the lines of the joints of the concrete layers. From this fact, the writer traced the cause of the deterioration to incomplete adherence of the successive layers, thereby permitting the percolation of sea water, and exposing the blocks to its chemical action. He reported what had happened to the Director of the Works, adding his personal opinion on the matter. Investigation was commenced by the Engineer-in-Chief, Mr. T. Okino, "Ingénieur des Arts et Manufactures," and other experts. Ultimately, it was concluded that the cause of the damage was no other than that which the writer had suggested, and the third method was considered to be an improved one, sufficient to remove the stated defect. The blocks which were made by the third method have, to date, presented no sign of deterioration.

Nevertheless, the writer was not content with this method; for he had reason to believe that there was still room for improvement, as he began to observe the percolation of water through blocks of this kind.

The test was made on 12 blocks. A vertical hole, 2 ft. deep and 4½ in. in diameter, was cut down into the middle of the upper surface of each block; a bamboo, 12 ft. long and 4 in. in diameter, with all joints removed, was erected thereon; the connection between them was made water-tight with cement mortar and a certain kind of bark; water was poured into the top of the bamboo until it was filled. The block was submitted to this test for five days, during which period the water was kept constantly at the same height by adding fresh water as soon as the water level in the bamboo was lowered, so as to keep a constant pressure upon the block. After one day, a spot or a band of water was observed on the lateral surface of three blocks. After three days, the same marks were observed on four other blocks. They did not appear to increase in number or size during the experiment. However, it was notice-

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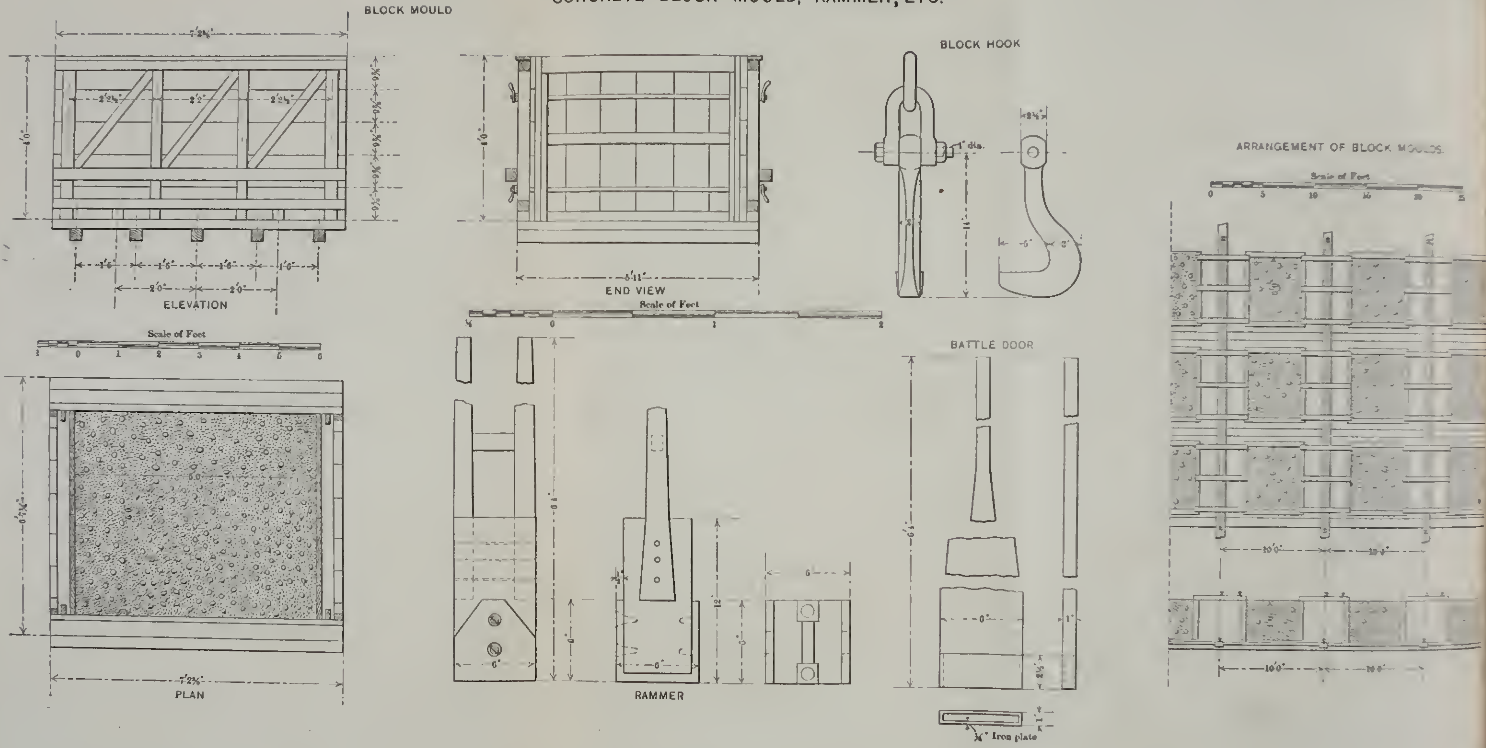
CONCRETE BLOCK MOULD, RAMMER, ETC.



able that they had appeared along the lines of joints of the concrete layers; this suggested that even the third method was not satisfactory to secure perfect impermeability between the layers. The remaining five blocks presented no signs of leakage, to the end of the experiment.

Thereupon, the writer made other specimens of blocks, giving every layer only one-half the former thickness, thus reducing it to about 5 in., and, at the same time, diminishing the period of ramming to 9 minutes. Then he submitted them to the experiment of the bamboo rods, as before stated. This time, no leakage of water was observed; so he concluded that the new method was more reliable than the former, and subsequently, the practical work was continued by this system until it was finished.

The writer has tried also to mix concrete by using a considerable quantity of water, *i. e.,* from 10 to 11% of the weight of the sand mortar, so that the concrete was brought to a gelatinous state. The concrete so formed was poured into the moulds; no ramming or patting was done, but it was forced with shovels into the corners and sides of the moulds after every charge. These blocks possessed perfect incorporation, nor has any deterioration been observed, due to the chemical action of the sea water. Moreover, the work was easy, and cost 43% less than by the ramming system. Yet, on the other hand, as water was used abundantly, and no special care was taken to remove it, the blocks became porous, and, consequently, less consolidated. The writer has found, comparing the average height of 200 blocks made by this system with blocks of the other systems, that the ratio was 1.032 to 1. Moreover, he observed very often that when the blocks collided with each other, or with any other hard substance, corners and edges were easily injured, whereas, the blocks made by the ramming system resisted well and suffered no damage. From these facts, it seems to be proved that they are not as strong, and, therefore, less reliable than the others. In addition to this defect, the writer was anxious because the proportions of the ingredients were not very rich for marine work; so he ceased to adopt this method.

As discussed in the foregoing pages, the writer believes that concrete blocks made by the ramming system are well fitted for

submarine works, even though the cement is used sparingly. He regrets only that he could not find the opportunity to prove more scientifically what he has said in connection with the experiments before stated.

TRANSACTIONS

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INTERNATIONAL ENGINEERING CONGRESS.

1904.

Paper No. 12.

HARBORS.

HARBORS ON LAKES ERIE AND ONTARIO.

By Dan C. Kingman, Maj., Corps of Engrs., U. S. A.

The title of this paper would apply to all harbors, whether natural or artificial, on both shores of Lakes Erie and Ontario, but it is intended to limit it to those harbors in the United States which are or have been under improvement by the General Government. It would be impossible to discuss in an intelligible way the progress which has been made in the last ten years in the improvement of harbors on these Lakes without first giving some consideration to the history of the harbors and the region which they serve; without offering some reasons why they were originally selected for improvement; and why the particular materials and methods which have been used for their betterment were selected. None of these harbors were originally planned to meet conditions at all comparable with those now existing. No one at that early time could have foreseen or imagined the enormous growth and development which has come to many of them, or the vast changes which have taken place in the means and methods of carrying on the commerce of the Lakes.

In 1820 the entire population of the eight States of the Union

bordering upon the Lakes, that is to say, New York, Pennsylvania, Ohio, Indiana, Illinois, Michigan, Wisconsin, and Minnesota, amounted to about 3 200 000. It now exceeds 32 000 000. In 1820 the entire population of the States of Ohio, Indiana, Illinois, Michigan, Wisconsin, and Minnesota, amounted to less than 800 000. It is now more than 18 000 000, and the increase in wealth and material prosperity has been in a vastly greater ratio. In 1820 such commerce as existed upon the Lakes, was carried on by small sloops or other sailing vessels. It was insignificant in volume and only amounted to what was necessary to serve the needs of a sparsely settled agricultural region. In 1902 the commerce of the Lakes demanded the service of steel steamers 500 ft. in length capable of carrying a cargo of 8 000 tons; and the receipts and shipments at the Lake Erie Harbors alone amounted to nearly 48 000 000 tons.

The Great Lakes of North America, penetrating far into the interior of a new continent, were early utilized as a pathway by explorers, for the purpose of trade with the aborigines, for warlike expeditions, later for the movement of settlers to the interior, and finally to serve the needs of a great inland commerce. The conquest and settlement of the continent spread from the Atlantic Coast toward the West, and for this reason Lake Ontario was the first of the chain of lakes to be utilized for purposes of navigation. At one time its trade exceeded that of all the other lakes combined, but as it is cut off from them by the Falls of Niagara its limit of usefulness was soon reached, and it is now of comparatively small importance for inland commerce. But when the obstacle of the Falls of Niagara is overcome by a canal of a capacity adequate to pass the largest vessels upon the Lakes with safety and promptness, Ontario will assume its rightful importance and will afford the eastern terminus for the Great Lake traffic.

In their natural condition these lakes offered very few safe and commodious harbors which were of sufficient depth to be used by vessels suitable for Lake navigation, and none at all which could be used by vessels of the present time. Shelter from storms and waves could be had within the mouths of the tributary streams, and sometimes in the bays or indentations of the shore line. These rivers and bays were generally deep enough for the small ships then in use, but their entrances from the lake were almost invariably

obstructed or completely closed by bars, formed by the joint action of waves and currents, at or near the line of the lake shore.

During the season when storms were prevalent on the Lakes the waves washed up the sand and gravel, and almost completely closed the mouths of the streams, but during the time of the spring freshets the volumes of water discharged by these streams were generally sufficient to cut their way through the bars and afford and maintain for a time channels deep enough to permit the passage of the small craft which were likely to use them. The bars which separated the bays from the Lakes were generally more permanent in character than those at the mouths of rivers, and channels, more or less tortuous and uncertain, always existed across them, through which the water ebbed and flowed as was necessary to maintain the level between the bays and the Lakes.

All of these channels were constantly shifting, were of uncertain and varying depth and liable to be obliterated temporarily by the action of storms. Such as they were, however, they were of necessity made use of by the first comers. Explorers, traders, and early settlers were too few in number, and their hold upon the country was too uncertain to permit them to undertake the permanent improvement of any harbor. It is told that when one of the first vessels came to Cleveland the crew had to be provided with shovels to dig a trench through the bar at the entrance to the Cuyahoga River through which they could draw their small vessel into the sheltered water within the river's mouth.

When the settlement of the region bordering upon the Lakes had advanced so far that the population produced something in excess of its own requirements and found it necessary to transport its products to market by water, the need of better harbors became imperative. Between 1820 and 1830 the improvement of a large number of harbors on Lakes Erie and Ontario was planned, and the actual work of betterment was undertaken at several of them. Where the proposed harbor was within the mouth of a river the first effort toward improvement was directed to doing away with the river bar, and this was effected by the construction of jetties extending outward into the lake so as to prolong the natural banks of the river and to confine its discharge to a channel of moderate width. The result was invariably to secure a deeper and better

entrance, and if the material was such that the current of the river could move it, the bar was completely washed away, and a channel secured and maintained between the jetties as wide and deep as the discharge of the river was capable of producing.

In the case of bays, in one or two instances, resort was had to dredging, but the dredging machinery of that early day was inefficient and the operation was tedious and expensive. In general, parallel jetties were constructed across the bar at or near the natural entrance, and the bar itself was protected by a similar structure built along its crest uniting the jetties with the shore. These "shore arms," as they were called, served to prevent other channels from cutting through the bar, and to confine the ebb and flow to a single well-defined entrance. The effect, of course, was to secure a better and deeper channel.

As the amount of money available for improving these harbors was small, it was necessary to construct such works as were required, of the cheapest material which would answer the purpose. Even if sufficient money had been available, it would have been impracticable at that time to procure skilled masons and quarrymen and the necessary machinery and appliances for the construction of stone jetties. The most available building material, therefore, was timber. The whole country was covered with forests, and logs could be had very cheaply and in unlimited quantity.

The portion of the jetties below the surface of the water was built of separate timber cribs. These were made of logs flattened upon two sides and notched together in the same way in which the houses or cabins of the time were built. The length of these cribs, which were of course rectangular in form, was that of a single log, 20 or 30 ft., and they were strengthened by a longitudinal wall, and one or two cross-walls, also built of logs. The logs were fastened together with wooden pins. The structures were floated into position, sunk upon the natural bottom of the lake, and filled with small stones gathered from the shores or from the fields. The cribs were sunk end to end along the line of the proposed jetty, and the superstructure was generally built of sawed timber and made continuous over a series of cribs. The side and cross-walls were generally 1 ft. thick and were carried up so as to give the jetty a height of 6 or 7 ft. above the ordinary level of the lake. The

superstructure was then filled with small stone and the whole was planked over to form a deck.

These structures possessed considerable strength, and as long as the timber of the superstructure was sound they could withstand the forces to which they were exposed. Unpainted and unprotected timber, however, decayed rapidly and the life of an ordinary superstructure was from 12 to 15 years. At the end of this time it required a complete renewal. As the work progressed the cribs were built of sawed timber in place of hewed logs, and iron drift-bolts were substituted for wooden pins. The submerged portion of the structure was not subject to decay, but as the cribs were not tight and were filled with loose stone, the water rose and fell inside of them as the waves came against them; and as this water in time of storms was filled with sand it scoured out and enlarged the openings particularly at the joints. The cribs were not built with tight bottoms, hence the stone escaped from them if the channel scoured out close enough to undercut them. This left them unsupported and occasionally was the cause of breaches being made through the jetties. In deeper water where the cribs were high, there was another cause of injury. The repeated shocks due to waves caused the small stone filling to exert a wedge-like action which spread the cribs apart at the corners.

In the later works the cribs have been made stronger by using vertical posts at the corners and cross-walls, and by bolting the structure together with strong screw-bolts. Tie-rods have also been used to prevent the walls from spreading, and within the last few years many of the cribs have been protected by sheathing them with hard wood plank 3 or 4 in. in thickness, placed vertically so as to protect the softer wood of the crib walls, and to afford it additional support and strength. This sheathing covers the openings so well that but little water flows in and out of the cribs. It also protects them from wear of ice and other floating bodies. The modern cribs have been made very much longer than those first used, some of them being more than 200 ft. in length.

Such were the reasons which led to the selection of certain harbors on Lakes Erie and Ontario for improvement, and such were the reasons that determined the choice of the particular materials and methods that were employed to secure the improvement desired.

As the population of the States bordering upon the Lakes increased, and the industries became more diversified, there was a rapid increase in the Lake commerce. This led to competition and the necessity for a reduction of freight rates, and, hence, to the use of a larger class of vessels. Larger vessels required deeper entrances to harbors, as the natural currents of the small streams and bays were seldom sufficient to secure or maintain of themselves a greater depth than 8 or 10 ft. Therefore, it became necessary to resort to dredging.

An Act of Congress as early as July, 1836, made an appropriation for a dredging machine for use on Lake Erie. As the amount appropriated was only $8 000, the dredge must have been a small affair. In 1852 three steam dredges were authorized for Lakes Erie. Ontario. and Michigan, respectively, at a cost of $20 000 each, including the equipment and the dump scows, or discharging scows, as they were called in the Act. This made it possible for the United States to do a portion of the dredging with its own machinery, but by far the greater part has been done by contract with private individuals or companies who furnished their own plant. A depth of 8 or 10 ft. was sufficient for the earlier commerce of the Lakes. This, however, was soon increased to 12, then to 16, and now a minimum of 21 ft. is required for all harbors used by the larger class of vessels.

The jetties, therefore, have long since lost their primitive function, which was an active one, namely, to scour out a channel by the natural force of the currents, and have become simply conservative in their nature, their office being to protect the artificial channels which have been secured and are now maintained by dredging. In the original designs the jetties were of necessity placed close together in order to confine the currents and wash away the bars, and as the streams were small, the distance between the jetties was from 120 to 250 ft., seldom exceeding the latter width. This made a very narrow entrance for a vessel to attain, particularly when driven toward the shore by a strong wind, and if it failed to make the entrance it was very apt to be wrecked on the beach.

In some of the earlier improvements the jetties were made flaring at the outer ends so as to give a funnel-shaped entrance and afford a wider opening for the convenience of vessels. No doubt, it had

FIG. 1.—WEST BREAKWATER, CLEVELAND HARBOR, SHOWING TIMBER SUPERSTRUCTURE IN A STATE OF DECAY, BEFORE IT WAS RENEWED IN CONCRETE.

FIG. 2.—WEST BREAKWATER, CLEVELAND HARBOR, SHOWING THE PARAPET AND BANQUETTE OF THE CONCRETE SUPERSTRUCTURE.

this effect, but it also afforded a wider opening for the entrance of waves, and caused the incoming wave to be heaped up at the gorge or narrowest portion and created at certain times a rough and dangerous channel; so that this style of construction was soon abandoned and the parallel jetties used instead. There was, therefore, great need at this time for harbors of refuge which could be entered with greater ease than these jettied channels, and where vessels could ride out a storm in safety and enter the commercial harbor when favorable conditions were resumed. This led to the planning of breakwaters for the most important harbors on the Lakes, that is to say, for Oswego on Lake Ontario, and Buffalo and Cleveland on Lake Erie.

The same reasons which caused the adoption of timber cribs filled with stone for jetty construction determined their use in the construction of the earlier breakwaters. The under-water portion of the breakwaters was generally built of timber cribs from 30 to 35 ft. square, divided into compartments by transverse and longitudinal walls, these cribs being sunk side by side along the line of the proposed breakwater. As in the case of the jetties they were filled with small stone, the superstructures were of timber and were continuous over a great many cribs, and generally continuous throughout the entire length of the breakwater. In the earlier forms the superstructure was simply carried up to a height of about 7 or 8 ft. above the Lake level and then decked over, but as the heavy seas came over them very freely so that a vessel could not lie alongside, the form was changed so as to make the outer part, or parapet, about 12 ft. high and the inner part, or banquette, about 5 ft. This gave a greater protection without increasing the quantity of material required for construction.

The same trouble was had with decaying superstructures and with damaged cribs due to sand and ice erosion and to the wedge-like action of the small stone filling.

The high vertical face which this style of breakwater presents to the lake, throws back the incoming sea in such a way that the reflected wave is superposed upon the advancing one in a manner to produce a very rough and irregular motion of the water surface, and this effect is extended to a considerable distance. Masters of vessels say that they can tell at night when they are passing opposite the

harbors of Cleveland or Oswego, several miles out in the lake, by the peculiar broken and disturbed seas which they encounter. This would indicate that the breakwater itself must be subject to very severe shocks, and indeed there is an abundance of evidence of a much more direct character that such is the case. The structures are racked and frequently suffer serious injuries from the force of the waves alone. To protect them from this action it has been proposed recently to deposit against them, on the exposed side, a mound of heavy rip-rap with a general surface slope of 1 on 2, and extending upward nearly to the top of the superstructure. Above the surface of the water, and for several feet below it, this large stone will be placed in the form of a rough pavement.

To meet the difficulty experienced by vessels in making the narrow entrance between the jetties at certain of the harbors, a plan has been devised for constructing two detached breakwaters converging toward the lake and having an opening between their outer extremities two or three times as great as the opening between the jetties, the entrance being placed on the prolongation of the axis of the channel and 1 000 or 1 500 ft. from the outer end of the jetties; and the gorge line of the breakwater to be situated at about the outer end of the jetties. This gives to the detached sections of breakwater a length of 1 500 or 2 000 ft. each, the angle which they make with the axis of the channel being varied from 30 to 45° as may be necessary to afford the maximum protection. Such works have been planned, and in part constructed, for the harbors of Lorain, Fairport, Ashtabula, and Conneaut, on Lake Erie. They will, when completed, give a much wider and safer entrance for the vessels, will permit them to enter, if necessary, at an angle with the axis, and give them space enough to straighten up before entering the narrow channel; and because the breakwaters diverge toward the shore they will give the waves coming through the entrance an opportunity to spread out and die down so as to leave an area of smooth water near the entrance to the jetties.

The great disadvantage of timber structures is the perishable nature of material when not entirely submerged. There is another disadvantage due to the continued increase in the cost of suitable timber, thus depriving them of the advantage of cheapness, which they originally possessed in a pre-eminent degree. The ordinary

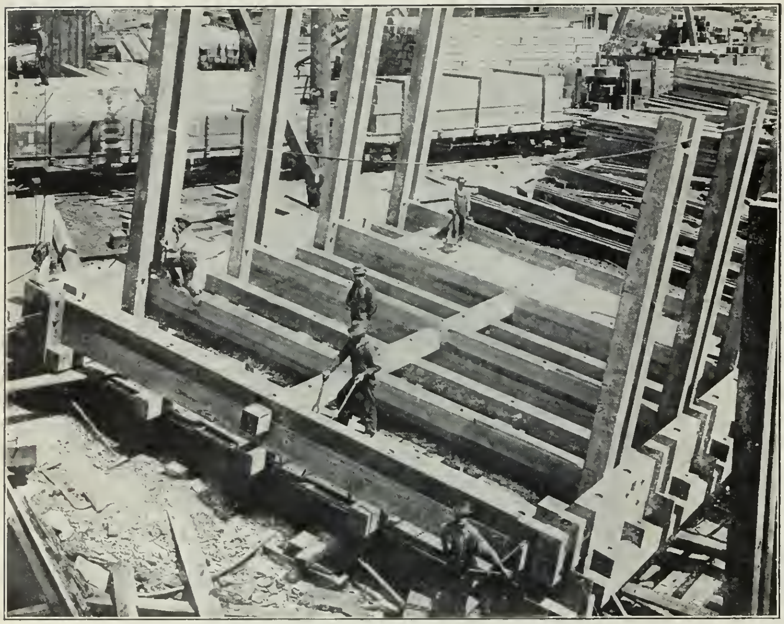


FIG. 1.—BUILDING A TIMBER CRIB FOR BREAKWATER CONSTRUCTION, SHOWING THE MANNER OF FRAMING AND FASTENING.

FIG. 2.—CLEVELAND HARBOR, OHIO. BUILDING A TIMBER BREAKWATER WITH SLOPING FRONT.

life of a timber superstructure is from 12 to 18 years, depending in part upon its exposure to the waves, and in part upon the excellence of the material of which it was originally constructed. On an average they should be renewed completely every fifteen years. The stone filling, of course, is available for the new structure, but the cost of rehandling it is almost as much as it is worth. To rebuild a timber superstructure costs about 30% of the original price of the entire structure. This means an annual fixed charge of 2% for maintenance of works of this character, not including extraordinary expenditures resulting from unusual accidents or injuries. This is a very heavy charge. It has been recognized from the first, and in the construction of the first breakwater at Oswego on Lake Ontario an attempt was made to build the superstructure of cut stone laid in cement and doweled together. However, the foundation afforded by the comparatively flexible timber cribs which yielded readily to the shock of waves, was so poor and so unsuited to carry a cut-stone masonry structure that the latter was soon shaken to pieces. Recently, concrete has been used extensively for the above-water portion of these works.

The timber-crib breakwaters at Cleveland and at Buffalo are now to a large extent equipped with concrete tops. The form given has generally been about the same as that adopted where timber was used, that is to say, in the case of breakwaters there has been a parapet and a banquette.

In preparing the cribs for a concrete top they are generally cut off at a depth of about 3 ft. below the mean level of the water surface, or well below the line above which the timber is subject to decay. Concrete blocks about 4 ft. wide, 4 ft. high, and 8 ft. long, or such other length as may be determined by the distance between the cross-walls, are prepared and placed upon top of the walls of the crib. Other blocks, if necessary, of lesser width are placed upon the center walls and cross-walls. The small stone filling is then carried up to within a few inches of the top of the blocks, and upon this surface the mass concrete is moulded in place. The blocks of mass concrete are generally from 20 to 30 ft. in length, the width being the full width of the crib. The blocks are separated from each other by a double layer of tarred paper.

In the Cleveland breakwater the mass concrete of the banquette

is 5 ft. thick, the parapet 5 ft. high, and the blocks are alternately 20 and 30 ft. in length. The weight of a block is from 300 to 450 tons, approximately, according to its length. They are built in such a way as to break joints with the cribs, most of which were 50 ft. long, and their mass and weight is largely relied upon to keep them in place. Notwithstanding their weight and the manner in which they are placed, they do work against and chafe each other during storms, due to the yielding of the crib substructure. It will probably be necessary to protect the whole with rip-rap on the lake side in the manner already described.

A few years ago a modification was made in the form of timber superstructure used in the extension of the east breakwater at Cleveland Harbor. It was carried up vertically for only 2 ft. above water level, then it was inclined at an angle of 1 on 2.5 until it attained a height of 10 ft. above the water surface. From that point it was horizontal until it met the harbor face which was vertical. All decking was of 12-in. timber throughout instead of plank, and the angle or knuckle near the water was protected from erosion and injury by iron straps 4 ft. long, 4 in. wide, and 0.5 in. thick, which were bolted vertically so as to cover the angle. It was considered that the protection in this form would stay on better and was not liable to such extensive injuries as an equivalent protection of iron plates. In observing this structure during storms, it is seen that the waves slide easily up the inclined surface and spill over the back of the structure into the harbor. The extensive reflex wave so noticeable in the vertical form is hardly perceptible in this case. The shock to the breakwater is evidently much less and the pressure which comes upon it is downward, and the resultant probably falls inside of the base. This form of superstructure which is so satisfactory in the case of timber is now being adopted where concrete is used on breakwaters. In the case of jetties the manner of building the concrete superstructure is similar, and the form is either the ordinary form used with the timber, or else with the gentle slope on the exposed side as in the case of breakwaters.

The first radical change in the materials and methods of breakwater and jetty construction on Lakes Erie and Ontario was suggested in 1895. A Board of Engineer Officers was appointed at that time to consider certain necessary improvements to the harbor

FIG. 1.—CLEVELAND HARBOR. A PORTION OF THE EAST BREAKWATER, SHOWING THE SUPERSTRUCTURE BUILT WITH A GENTLE SLOPE ON THE EXPOSED SIDE, UP WHICH THE WAVES CAN SLIDE WITHOUT SHOCK.

FIG. 2.—HARBOR OF FAIRPORT, OHIO, LOOKING OUTWARD TOWARD THE LAKE FROM THE OUTER END OF THE JETTIES, SHOWING VESSELS IN THE ICE-DAM WHICH RESULTED FROM THE FLOOD OF 1904.

of Buffalo. It was proposed to extend the breakwater about 12 000 ft., so as to afford a much larger sheltered area for the convenience of vessels and the protection of wharves. The Board recommended a rubble-mound breakwater similar in construction to a number of stone breakwaters which had been successfully built on the Atlantic Coast. It was proposed to build up first a mound of small stone to a height of about 12 ft. below the mean lake level, and upon this to build, with large stone from 4 to 10 tons in weight, up to a height of about 10 ft. above that level. The large stones were to be deposited along the axial line of the proposed structure, with side slopes as steep as the material would naturally assume. It was expected that the action of storm waves would reduce the height and flatten the slopes to a position of ultimate stability. It was recognized that the structure would have to be recapped, probably more than once, with large stone until it remained permanently at a height of about 10 ft. above the water at ordinary lake level. The advantages claimed for a breakwater of this kind were: That nothing placed in it was lost; that no conceivable injury to it could do more than reduce its height somewhat; that it must always be left in the best possible condition for repairs; and that a time must certainly come when repairs of any kind would cease to be necessary.

When the breakwater was constructed the recommendations of the Board were not fully carried out, but under proper authority a number of modifications were made. The core, or heart, for reasons of economy, was made of coarse gravel which was carried up to a height of about 10 ft. below the water surface. This core was covered with a very thick layer of stone of quarry-run sizes, that is to say, of the irregular sizes and shapes that could be most cheaply and readily obtained from the quarry. It was of large and small stone mixed together. From a point 15 ft. below the water surface on the lake side a paving of heavy blocks of stone was carried up over the structure, and down on the harbor side to a level 10 ft. below the water surface. To this portion of the structure a regular and uniform profile was given and the pavement made the slopes smooth and even, and the structure presented a fine appearance. From 15 ft. below water on the lake side up to 3 ft. above, the pavement was given a slope of 1 on 2.5; thence upward to the top it had a slope of 1 on 1.22. The top was horizontal and 14 ft.

wide, and 12 ft. above the ordinary lake level. On the harbor side the slope of the pavement was 1 on 0.7.

A rubble-mound breakwater has been adopted for the proposed improvement of the harbor of Cleveland, Ohio, which will involve the construction of nearly 20 000 lin. ft. of this structure. Rubble mounds are also used for the detached breakwaters at Ashtabula, and Lorain. The cross-section of the Cleveland breakwater differs somewhat from that of Buffalo. In the construction parallel ridges of large stone are first placed at a proper distance apart to outline the base of the structure. They are carried up to a height 20 ft. below the water surface. Their inner slopes are then covered with a very thick layer of shale, coarse gravel, or furnace slag, and the space between these slopes is filled with a core of lake sand. A second set of ridges conforming to the slopes is then built of very heavy stone, and the core is carried up to the proper height. Upon this a thickness of about 9 ft. of small stone is placed, and this in turn is covered by a smooth pavement of heavy blocks about 3 or 4 ft. thick. The pavement extends from 12 ft. below water on the lake side to 8 ft. below on the harbor side. In the profile the slopes are as follows: On the harbor side a uniform slope of 1 on 1.3; on the lake side from the superior crest down to a depth of 12 ft. a uniform slope of 1 on 2; thence to the bottom a slope of 1 on 1.5.

At Ashtabula and in the west breakwater at Lorain the construction will be similar, but as the structures are in shallower water the sand core is not used. The pavement presents a smooth surface and, therefore, a fine appearance, and as the stones which compose it are large and carefully placed they are not at all likely to be moved or disturbed by waves or ice. The style of construction, however, has this disadvantage that if the pavement is broken the injury is likely to be wide spread, and if the structure settles irregularly and unequally it is more difficult to bring it back to grade than in the case of a simple rubble mound.

The most recent design for a rubble-mound breakwater is that which has just been approved for the east breakwater at Lorain. This has a core of shale, a covering of quarry-run stone, and from a depth of 12 ft. below the surface, a covering 12 ft. thick of large blocks of stone from 8 to 12 tons in weight. These stones are simply deposited in place and are not laid as a pavement. More

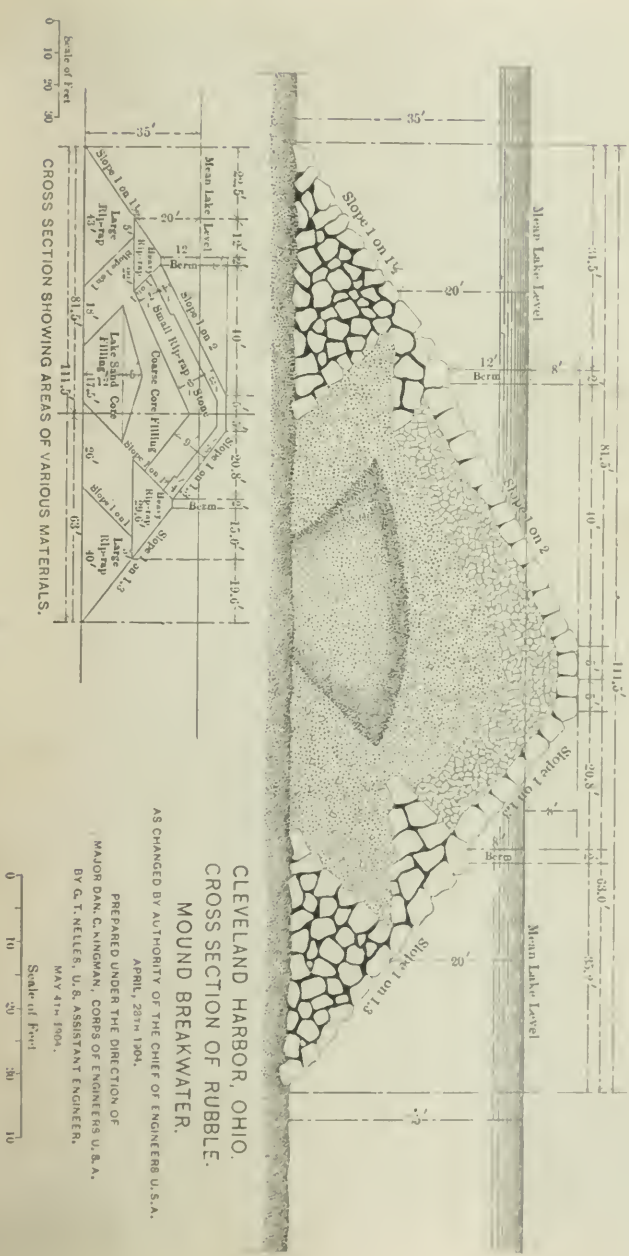

CLEVELAND HARBOR, OHIO.
CROSS SECTION OF RUBBLE.
MOUND BREAKWATER.
AS CHANGED BY AUTHORITY OF THE CHIEF OF ENGINEERS U.S.A.
APRIL, 28TH 1904.
PREPARED UNDER THE DIRECTION OF
MAJOR DAN. C. KINGMAN, CORPS OF ENGINEERS U.S.A.
BY G. T. NELLES, U.S. ASSISTANT ENGINEER.
MAY 4TH 1904.
Scale of Feet
Mean Lake Level
Slope 1 on 1½
Slope 1 on 2
Slope 1 on 1.3
Berm
CROSS SECTION SHOWING AREAS OF VARIOUS MATERIALS.
Large Rip-rap
Heavy Rip-rap
Small Rip-rap
Coarse Core Filling
Lake Sand Core Filling

Fig. 19.

care is taken to bed and bond them well than to make them conform to any particular cross-section. It is believed that this will prove the cheapest form of rubble-mound construction that has yet been tried.

As long as the jetties were parallel and close together it is evident that a rubble-mound construction would have been open to many objections. Pieces of stone might wash down into the channel and vessels might be injured by striking them or by striking against the rough and irregular slopes. But now that jetties are no longer used as active works to secure and maintain channels, but have for their office simply to protect channels which have been deepened by dredging, there is no longer any necessity for keeping the jetties parallel or for building them close to the sides of the cut. It is possible, therefore, to give them almost any position which will break up the waves and prevent material from being washed into the channel. In a new jetty recently planned for improving the harbor at Huron, Ohio, it was proposed to use a rubble-mound jetty terminating at its outer extremity in a timber-crib pier head with a concrete superstructure placed at a proper distance from the west jetty to afford the desired width of entrance. From the pier head the jetty extends toward the shore, diverging from the channel so as to finally reach the shore 1 000 ft. or more from the other jetty. It is believed that this will fulfill every purpose required of it, and at the same time will shelter a considerable length of lake shore; and that it may be built upon the natural bottom of the lake provided no dredging is allowed nearer the toe of the slope on the harbor side than ten times the depth to which it is proposed to dig. The cross-section of this jetty has a very simple form. The structure has a core of small stone, shale or sand, and is built up in the main of large blocks of heavy rip-rap without any effort to present a smooth surface or to conform strictly to a fixed profile.

Now that materials which are not subject to decay have been adopted for jetty and breakwater construction, and a style and method of construction which produce structures strong enough to resist the forces to which they are exposed are used, it follows that all work of this character proposed for the improvement of harbors on these lakes will finally be completed, and that thereafter it will only be necessary to maintain the improvement which has

been secured. It is evident that the things of real value to navigation are the artificially deepened channels and areas. All the other works are simply designed to protect or shelter them, and, as the protection and shelter cannot be made perfect, dredging will continue to be necessary at all of the harbors.

The type of dredge heretofore used on the Lakes has been the "dipper" dredge, which has gradually developed from a crude affair, operated by the power of a single horse, to a machine which can handle a dipper of 8 or 10 cu. yd. capacity. The dipper dredge is as nearly a universal machine as any that could be devised. It can excavate anything from soft rock to mud or sand, but it is an unseaworthy style of craft and is dangerous to take out a long distance in the lake. Moreover, from its method of operation it cannot dig when the water is rough, and hence a great deal of time is lost to these dredges when they are required to work in an exposed locality.

The United States is now building a special dredge for use on Lake Erie. It is of the sea-going suction type. Its capacity should far exceed that of the best dipper dredge, and its construction will enable it to work in exposed localities at all times except during violent storms. Great economy and convenience will undoubtedly result from its use.

The harbor of Toledo, on Lake Erie, is an exception to the ordinary jettied harbors. It is within the mouth of the Maumee River, and the improvement is effected solely by dredging. No jetties, breakwaters, or similar works are resorted to.

The plan of improvement which is now in course of execution provides for the excavation of a channel 400 ft. wide and 21 ft. deep through Maumee River and Bay. The total length of this channel is about 15 miles, of which 8 miles are in the bay and 7 miles within the mouth of the river, and the work will require the removal of 8 000 000 cu. yd. of material. The channel in the bay is excavated in a perfectly straight line from the mouth of the river to the 21-ft. contour in the lake. The natural depth of the bay, for a greater portion of this distance, is 10 ft. or less. The material in which the cut is made is a moderately stiff clay. Observations made upon the unfinished work indicate that a considerable amount of dredging will be necessary to maintain the channel. It seems probable

that the average annual amount will not be less than 300 000 cu. yd.; but the removal of this quantity of silt with a suitable dredge will cost very much less than the annual interest upon cost of parallel jetties of this length.

It must be admitted that a small river or creek does not and cannot furnish the most favorable location for the development of a large commercial harbor. The ground bordering the stream is often unfavorable for supporting heavy structures. The valley of the stream is apt to be narrow and contracted, and the stream itself, while it may prove of some assistance at the outset, is a weak ally, and in the end, a source of danger and expense. The periodical freshets which occur in it, the ice gorges which occasionally form in a cold climate, and the large amount of silt which it annually discharges, are sources of difficulty and danger which lavish expenditure may mitigate but cannot abolish. It is believed, if it were now a question of selecting a site upon the shore of either of these lakes for a great harbor where a large city should be built and an extensive commerce developed, that the mouths of creeks and rivers should be carefully avoided. It would be best to select a point upon the straight shore of the lake where the bank was not too high, where the foundation was good, and where deep water comes in near to the shore. There, a strong and durable breakwater should be built inclosing an ample area and provided with safe entrances and exits. Under the shelter of this structure, wharves should be constructed and railway tracks should lead to them directly from the interior. Such an arrangement would lend itself best to an extensive commerce and would be susceptible of development and expansion to conform to the growth of trade.

The harbors of Lorain, Cleveland, and Fairport, are all at the mouths of small rivers, and observations have been made covering a considerable period of time to determine the regimen of these streams. The areas of the drainage basins of the streams amount to 550, 972 and 759 sq. miles, respectively. The mean annual discharge of the three streams is 648, 1 345 and 1 237 cu. ft. per sec. The discharge from day to day is subject to wide variation, the flood being ten or twelve times as great as the mean.

Daily sediment observations upon these three streams covering a period of about a year, during which no abnormal conditions

occurred, indicated that the Black River brought down annually 106 000 yd. of sediment, the Cuyahoga River, 193 000 yd., and the Grand River, 157 000 yd. These are large amounts of material to be deposited in artificially deepened areas or in a shallow lake near the entrances of harbors. It may be stated, however, that while this annual tribute must have an important effect in time, its effect from year to year is less than one would suppose. Nine-tenths of the amount of silt is discharged during perhaps half a dozen freshets each one of but 2 or 3 days' duration, and during these freshets the velocity of water is sufficient to cause a strong current to be directed far out into the lake. The exact course pursued by this current after it leaves the jetties is determined or materially modified by the accidental direction and force of the shore currents at the time. Hence, the greater part of the material is scattered over a wide area.

It is worthy of note that in the case of these three rivers, during the year that they were under observation, the quantity of sediment discharged seemed to be a constant per square mile of territory in the drainage basin, amounting in each case to about 200 cu. yd. per sq. mile. It is recognized that this may be a mere coincidence and that it certainly would be materially modified in a year of abnormal freshets, but if it indicates an average which holds good throughout the Erie water-shed of the State of Ohio, then this State alone delivers an annual tribute of 2 500 000 cu. yd. of silt to Lake Erie, and nine-tenths of this quantity enters the lake through the mouths of creeks which have been improved as harbors.

If freshets occur when the streams are covered with ice, conditions arise which are dangerous and destructive, not only to wharves and shipping, but to the dredged channels and areas as well.

The first half of the winter of 1903-04 was very cold, and the ice upon the bays and rivers became 18 in. or 2 ft. in thickness. It even formed on Lake Erie itself along the shore, extending outward as far as the eye could reach. The ice in the lake was repeatedly broken up by waves as it formed, and the ice floes were forced under each other until the whole became exceedingly thick and solid.

In the latter part of January the weather suddenly moderated. The temperature rose to a point 15 or 20° above freezing. At the same time a copious warm rain set in. The water-shed was covered

with deep snow which was melted by the rain and added to the water of the rainfall. The ground was frozen and almost the entire quantity of water found its way quickly into the streams, producing sudden and violent freshets. The ice was broken up and swept down the streams, repeatedly lodging and forming dams which backed up the water until the head was sufficient to sweep them away only to reform further down stream.

In the Grand River, which enters Lake Erie at the harbor of Fairport, the maximum discharge amounted to nearly 16 000 cu. ft. per sec. Ice dams were repeatedly formed, and when the moving ice reached the lower part of the river it came as a solid wall, rising 10 ft. or more above the surface of the water. The vessels and small craft in its way were torn from their moorings and swept out into the lake. When the moving ice met the solid ice at the entrance of the harbor it formed a dam of unusual strength and solidity. It backed up the water to a height of 8 or 9 ft. so that it flowed out over the top of the jetties. These structures were not designed to act as dams, and the water under this head found its way beneath them, scouring out the bottom and causing them to settle unequally and disturbing their alignment. The water finally burrowed under the ice dam scouring out a channel and depositing the eroded material out in the lake beyond the dam, so as to form a bar, upon the crest of which there was but 6 or 8 ft. of water. When the ice disappeared the harbor was thus closed to commerce. To restore the channel dredging was resorted to, and it required the removal of 30 000 yd. of material to accomplish this result.

A freshet occurred at the same time in the Cuyahoga River which enters Lake Erie at Cleveland, Ohio. The maximum discharge amounted to 28 000 cu. ft. per sec. The lower part of this river was free from ice because the city had caused tugs to move up and down over it continually to prevent the solid ice from forming. Hence, there were no ice dams, but the discharge was so great that the river had a fall of 14 ft. in a distance of 3.5 miles within the city limits. The current developed was very great and vessels were torn from their moorings and three large ships were swept down the river and lodged against the Superior Street Viaduct which is the principal public bridge across the river. The bridge was thereby greatly endangered, the channel was blocked, and the vessels were

not extricated from their perilous position for several days. The swift current scoured out the bed of the river in some places and caused extensive deposits in others, and not less than 250 000 cu. yd. of sediment and filthy sludge which had been deposited by the city sewers on the bottom of the river was swept out through the jetties in a period of about ten days. The volume of water passing during that period was about 9 000 000 000 cu. ft. A portion of the sediment was deposited in the outer harbor, but the greater part of it was swept into the lake. The deposit in the harbor, however, was sufficient to require extensive dredging to restore navigation.

In the Black River which enters Lake Erie at the harbor of Lorain, a similar freshet occurred causing extensive ice dams which raised the water not less than 12 ft., so that the low land was inundated and the river found temporarily a new outlet to the lake. Bridges were damaged and vessels were swept out and stranded in the meadows.

The Maumee River which enters Lake Erie at Toledo, Ohio, was likewise in a state of flood, and formed an ice dam in the bay about a mile beyond the river mouth. This dam was not less than 600 ft. thick and solid to the bottom of the bay and the dredged channel. It backed up the water until it attained a height of 11 ft. above the mean stage. Wharves and railway tracks and the lower portions of the city were submerged, bridges were destroyed, and vessels swept from their moorings. The ice dam was not broken for more than two months, but channels were formed under the dam through which the impounded water gradually escaped.

Although the action of floods and ice may at times be very injurious and destructive to artificially deepened channels and areas, by far the most persistent cause of deterioration is due to the movement of the beach sand under the combined action of waves and currents.

The existence of a sandy beach implies the existence, not far away, of a source of supply of beach-forming material. This is usually a high bluff bank or headland that is being gradually undermined and washed away, the material of which is assorted and transported by the water so as to form the beach in the first place and to maintain it, and perhaps augment it, thereafter. The beach sand is moved in two different ways: First, by the direct action of the

waves breaking upon the shore; and secondly by the action of the littoral currents.

When a wave advances into gradually shoaling water so as to feel the retarding effect of the bottom and the diminishing depth, the axis of oscillation of the particles of water ceases to be vertical and becomes inclined toward the shore, the shoreward face of the wave becomes steeper and steeper and finally breaks, and the water flows rapidly up the beach and immediately flows back again. The motion of translation of the water as the depth diminishes becomes sufficiently great to sweep the sand along the bottom, to carry it up the incline, and, as the wave returns, to carry it, or the greater part of it, back again. If the crest of the in-coming wave is not parallel to the shore line the sand is pushed up the beach obliquely, and as the reflow is nearly normal to the shore the sand is advanced by each successive wave in the direction along the shore in which the waves are running. Another storm in the opposite direction may move it back again, but if, as is usually the case, the prevailing storms are from one direction the resultant movement of the sand will conform.

Upon the Lower Lakes the prevailing storms are from the westward, and as a result there is a constant movement of sand toward the east. If this movement is arrested by a wharf, jetty, or other structure extending outward from the shore, the sand will gather against it, and the shore line will advance accordingly.

The second mode of sand movement results from the action of the shore currents. These currents on the Lakes are not sufficient of themselves to pick up the material from the bottom and move it along; but during storms, when the waves are breaking upon the shore, they stir up the sand at the source of supply as well as upon the beach itself, lift it up and hold it in suspension for a time, the finest and lightest particles being held the longest. If, at this time, there happens to be a current along the shore, or outward toward deeper water in the lake, the material so suspended will move with the current until it gradually sinks and comes to rest upon the bottom. This action may make itself felt in much deeper water than that of the waves alone.

On the Lower Lakes the direct action of the waves produces little or no motion of sand in depths of water greater than 12 or

15 ft., but the currents may carry the suspended material into any depth, depending upon their velocity and direction and the length of time that the silt can be floated.

At the harbor of Fairport on Lake Erie in the State of Ohio, the sand movement is perhaps as marked and extensive as at any harbor on the Lower Lakes. For this reason it is a particularly favorable place to study the sand movement. The earlier maps are not extensive enough or in sufficient detail to furnish data for a complete and satisfactory comparison, but such a comparison as it is possible to make indicates that not less than 500 000 cu. yd. of sand have accumulated in the last 48 years in an area bounded by the original shore line, the west jetty, the 20-ft. contour in the lake, and a straight line parallel to the jetty and 1 000 ft. west of it. It was possible to extend the comparison only 1 000 ft. from the jetty on account of the limited area covered by the old map, but the actual amount of sand arrested by the jetty must be three or four times as great as this. This represents the net result of the sand movement within this area, but it by no means represents the aggregate movement. The conditions are always such as to cause any excavation or artificial deepening to be quickly filled. A single storm, in November, 1901, caused a fill of not less than 20 000 cu. yd. to take place in an area 500 by 250 ft. in the channel immediately beyond the outer end of the jetties.

A number of observations have been made at Fairport to determine the velocity and direction of the shore currents, and the amount of sediment carried in suspension. The prevailing direction of the currents was found to be from west to east, generally parallel to the shore, but moving outward to pass around the end of the jetties.

In making the current observations, weighted rods nearly equal in length to the depth of the water were used. They were submerged for about nineteen-twentieths of their length so that the wind could have very little effect upon them. At the time the current measurements were made, observations were taken to determine the amount of sediment in suspension in the water. Samples of water were taken at a number of different buoys properly located in the direction of the current. These samples were taken about 2 or 3 ft. above the bottom, and the amount of sediment in a given

volume of water was afterward determined by filtering and weighing. The results showed that the currents might be as great as 1 mile an hour and that the sediment in suspension always diminished as the water moved outward from the shore. The loss of suspended sediment was sufficient to account for a fill across the channel beyond the end of the jetties as great as 0.4 ft. in 24 hours during a storm of very moderate violence.

These observations were not taken under conditions to give maximum results. A small boat was used for placing and picking up the floats and for securing samples of water, and it was not possible to send it out at the time of a very severe storm. If it had been possible, no doubt, greater currents would have been observed, a much larger amount of sediment would have been found in suspension, and its effect would have extended very much farther outward into the lake.

It is evident, therefore, that the forces of Nature are constantly at work to destroy and obliterate the artificially deepened channels and areas which form such an essential part of the harbors of the Lower Lakes, and that no permanent structures can be devised that will adequately protect and maintain them. Periodical dredging will always be necessary to preserve the required depths.

The wisdom and necessity of keeping in a state of preparedness, a dredging plant of sufficient capacity, seaworthy in construction and design, rapid in action, and economical in operation, is therefore too obvious to require demonstration.

The suitability of any harbor for the purpose for which it is designed can best be determined by considering the kind of vessels which make use of it. In the earlier period of the navigation of the Lower Lakes, sailing vessels were exclusively used for the transportation of freight. The fore-and-aft rig was generally adopted and the two- or three-masted schooner was the type of the Lake freighter. These sailing craft found difficulty in entering and leaving the jettied harbors and in making their way through the narrow bodies of water which connect the Lakes. Steam tugs were therefore early resorted to to assist them in these places, and the enterprise of the tug owners led them to cause their tugs to cruise far out in the lake to pick up the tows. When the winds were adverse or when there was no wind at all, the tug sometimes pro-

FIG. 1.—CLEVELAND HARBOR, SHOWING JETTIES AT THE MOUTH OF CUYAHOGA RIVER, AND THE MAIN ENTRANCE IN THE BREAKWATER.

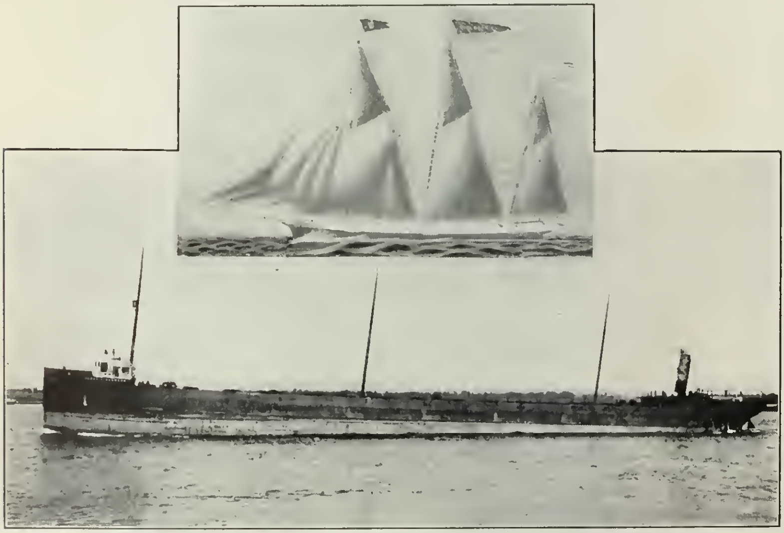


FIG. 2.—THE STEAMER "ISAAC L. ELLWOOD."
Gross Tonnage, 5 904; length, 478 ft.; breadth, 52 ft.; depth, 30 ft.; built in 1900.
THE SCHOONER "ALVA BRADLEY."
Gross Tonnage, 934; length, 192 ft.; breadth, 32 ft.; depth, 20 ft.; built in 1870.
These are photographed to the same scale, and show the change in the standard Lake Freighter in 30 years.

ceeded with the vessel to its place of destination. This service proved so expeditious, so convenient, and so advantageous to commerce, that a system of towing was soon developed. The small tug gave place to a steam vessel large enough to carry a cargo itself, and this vessel towed a number of schooners, one behind the other, with long tow-lines between.

When this system became well established the sail area of the schooners was greatly reduced. Their topmasts and small sails were done away with, leaving them only the lower sails, which were used under favorable conditions to assist the tow-boat, or to give the crew some control over their vessel in case the tow-line parted. Afterward, a special class of vessels of large size was built to be towed by steamers, and the peculiar shape of hull, both for the steamer and the consort, known as the whaleback, was developed and quite extensively used. The pointed ends and rounded deck which gave this vessel its name were thought to have some special advantages. But this style of construction was soon abandoned, and no more of them are being built. The ordinary form of a ship's hull is now regarded as the best.

Very large cargoes could be carried by the tow-boat and its consorts combined, and great economy of transportation was effected thereby, and for a time it was in high favor. But the tendency now seems to be to place the same motive power in a larger hull and to dispense with the consorts. Two or three boats combined require more wharf room than a single large vessel. They are difficult to handle in narrow waters and the condition of the consorts is perilous if the tow-line is parted in a storm. The dividing up of the crew is disadvantageous, and there is likely to be more delay in loading and discharging several small vessels than one large one. It seems probable now that the future freighter of the Lakes will be a very large propeller of moderate draft and power, that it will carry the cargo in its own hold and will not be embarrassed by other vessels towing behind it.

In the improvement of the harbors on the Lakes the General Government of the United States has constructed jetties and breakwaters, and has secured and maintained a sufficient depth of water in the channels and approaches and in the anchoring grounds. But the General Government has not undertaken the construction of

wharves or docks, or the land approaches leading to the same; nor has it provided warehouses or wharf machinery. The construction of these things has been left to those more immediately concerned.

In all the Great Lake harbors these utilities are built, owned, and operated by private individuals or incorporated companies. The level of the Lakes is so nearly constant that inclosed docks are nowhere necessary for the convenience of loading or discharging vessels. The wharves are simple affairs revetted with timber cribs or sheet-piling. The approaches to the wharves are generally controlled by the railway systems of the country, and abundant trackage is furnished so as to place the trains conveniently alongside the ships. Elevators of the best modern construction and design are provided for the handling of grain, and such warehouses, as are necessary, are supplied for the shelter of package freight. Iron ore and coal, the two principal commodities received and shipped at the harbors of the Lower Lakes, need no shelter, but the machinery for handling ore and coal is admirable in its rapidity of action and its economy and labor-saving efficiency.

The Hulett automatic ore unloaders which are in use at the Harbor of Conneaut, Ohio, are considered the best type of unloaders on the Lakes. These machines at times attain a speed of 500 tons per machine per hour. Four of them are installed at Conneaut and can work together on a single vessel, making a total maximum tonnage going out of the boat of 2 000 tons per hour. Some of the largest ships have been completely unloaded by these 4 machines in less than 4 hours.

The McMyler car-dumping machines which are in use at many of the harbors on the Lower Lakes are admirable examples of machinery for handling coal. The earlier machines were of the "end-dumping" type. With these the car is hauled up an inclined girder which is pivoted near the upper end and projects over the vessel to be loaded. The coal car is drawn to the upper end of this girder by means of a haulage engine in the machine. The car is then clamped and the end gate removed, and the girder with the car upon it is tilted to an angle of about 45° so that the coal is poured into the hatch of the vessel. The objection to this machine is that it requires a particular kind of coal car equipped with an end gate. It has been largely superseded by the side-dumping type. This machine is designed to deliver coal from all sizes and kinds of cars

used in the coal trade to all sizes and kinds of vessels. The machinery is designed to handle the coal with a minimum amount of breakage, and to accomplish this the coal slides from the car to the vessel, all vertical drops being avoided. The machine consists of a frame of structural steel about 75 ft. high, 52 ft. long, and 40 ft. wide, and of the necessary machinery for hauling the car into the machine, raising and turning the car over, and for adjusting the different parts to suit the various sizes of cars and vessels.

The operation is as follows: The loaded cars are placed in the yard and are conveyed to a point near the machine one at a time, either by gravity or wire-rope haulage. The car is then pushed up an 8° incline into the machine by a haulage engine operating a small push-car, on a narrow-gauge track, which drops into a pit at the foot of the incline to allow the car to pass over it. After the car is placed in the machine it is clamped automatically and raised to the elevation of a triangular apron when it is turned over and the coal flows down the apron into a telescopic chute which discharges it into the hold of the vessel. The apron may be raised or lowered to allow vessels to pass under it and to accommodate ships of different size. The telescopic chute is designed to swing across the hatch, and a trimmer is provided to deliver the coal under the decks between the hatches. This trimmer consists of a spout attached to the lower end of the chute and arranged to turn horizontally and deliver the coal in any direction desired. The machines are operated either by steam or electricity, and capstans are provided on the docks for moving the vessels promptly. These machines will handle from 20 to 25 cars per hour, delivering 1 000 or 1 200 tons of coal to the vessel in this time.

It must be admitted that the control of these harbor facilities by private companies is not without certain advantages. These companies can always command the best talent to design, erect and operate their machinery, and they have the power to act quickly, and are able to tear out and throw away good and serviceable machinery as soon as something better is discovered. On the other hand, private companies are working for the benefit of their stockholders. They are not interested in reducing the burdens of commerce any further than may be necessary to enable them to secure the business. They are not concerned with the development of any trade but their own, and they are apt to be more anxious to exclude

or destroy rivals than to build up the general commerce of the country, or to promote the general welfare. It is certain that the cities and towns at the different ports upon the Great Lakes will sooner or later wake up to their own interests and will follow the example of the great seaports of the world, and acquire the ownership of their water fronts, build the wharves and land approaches, control the warehouses and wharf machinery, and operate the same for the public good, for the development of the general commerce and with equal justice to all the interests involved.

To sum up in a few words the progress that has been made in the theory and the practice of improving the United States Harbors on the Lower Lakes in the past ten years, it may be stated that when timber cribs are used for the construction of jetties and breakwaters, they are made longer and stronger than formerly, and screw-bolts and steel tie-rods are more extensively used to hold the walls together, and the crib-walls are sheathed with hard-wood plank. Timber superstructures are being generally replaced by concrete moulded in place; and the concrete is often given a profile that offers an inclined surface up which the waves can flow, without shock. Rubble-mound structures are being used instead of timber cribs for breakwaters, and even for jetties where the local conditions are such as to permit it.

Periodical dredging is now known to be necessary to maintain the harbors, and the office of jetties and breakwaters is limited to affording protection and shelter.

Hence, it is recognized that jetties need no longer be parallel, or be built close to the sides of the channel. Adequate dredging plants are being acquired, to be owned and operated by the United States, for the improvement and maintenance of the harbors.

Finally, it is admitted that whereas in former times when ships were cheap and small, and harbor building and harbor improvements were comparatively expensive, the ships were made to suit the ports; but now, with the enormous and costly vessels that the requirements of economical transportation demand, the harbors must be made to suit the ships, and the United States is making liberal appropriations and expenditures to widen and deepen the channels and increase the capacity of harbors to keep pace with the growth and development of the Great Lake commerce.

TRANSACTIONS

AMERICAN SOCIETY OF CIVIL ENGINEERS.

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Paper No. 13.

HARBORS.

HARBORS ON LAKE SUPERIOR, PARTICULARLY DULUTH-SUPERIOR HARBOR.

By David DuB. Gaillard, Maj., Corps of Engrs., U. S. A.

Physical Characteristics of Lake Superior.

Lake Superior is the greatest expanse of fresh water on the globe.

Its length, along the steamer track from Duluth, Minn., to Pt. Iroquois, Michigan, is 383 miles, and its greatest breadth is about 160 miles. Other dimensions and characteristics are as follows:

Area of water surface of lake..................	31 800	sq. miles.
Area (land) drained..........................	48 600	" "
Total area of basin..........................	80 400	" "
Average rainfall per annum..................	28	in.
Maximum depth of lake......................	1 012	ft.
Mean surface above mean tide at New York City.	602.28	"
Mean level above mean level of Lake Huron...	20.87	"
Low-water datum for harbor improvements, above mean tide at New York City..........	601.75	"
Discharge of outlet, St. Mary's River, as measured at mean stage of Lake Superior........	60 600	cu. ft. per sec.

The water levels of Lake Superior during the navigation season, May to December, inclusive, as determined from daily gauge readings at Marquette, Mich., are as follows:

Mean level, 29 years (1872-1900).....	0.67 ft. above low-water datum.
Highest monthly mean level (August, 1876)..........................	2.18 ft. above low-water datum.
Lowest monthly mean level (May, 1879)..........................	0.74 ft. below low-water datum.

The average date of opening of navigation at St. Mary's Falls Canal, Michigan, is April 27th, and the average date of closing of navigation at the same place is December 2d.

Owing to its great depth, the middle part of the lake never freezes, although at times a temperature as low as 40° fahr., below zero, has been recorded along its shores.

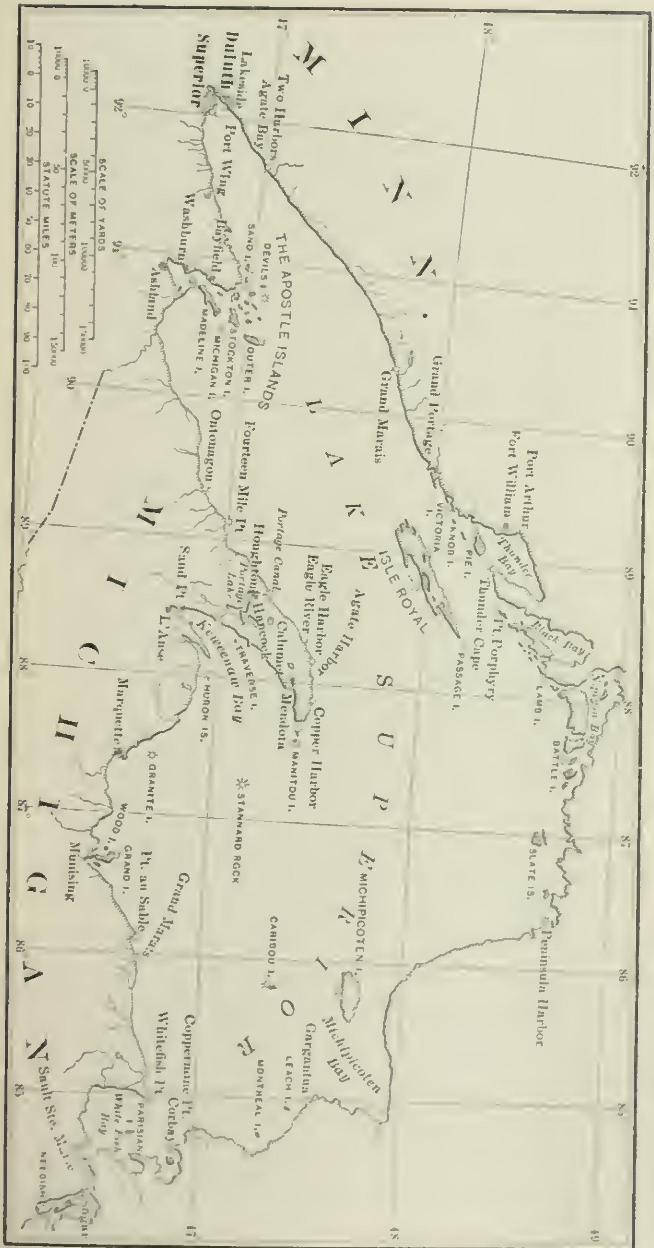
During the unusually severe winter of 1903-04, the ice at most points on the lake extended out from shore beyond the range of vision, and it was supposed by many that the entire lake was frozen over, but open water in mid-lake was at all times visible from a few high points on shore.

The maximum thickness of ice in the harbors on Lake Superior varies from 12 or 15 in. to 3 or 4 ft., depending upon the severity of the winter and upon the particular locality considered.

The average date of closing of the different harbors by ice varies from November 19th to January 11th, and the average date of opening varies from April 20th to May 5th.

In addition to its size, Lake Superior is characterized, when compared with the other Great Lakes, by greater depth; greater elevation of its surface above sea level; its high and rocky shores, especially on the north; its irregularity of outline (see Fig. 20); the lower temperature of its waters, and greater prevalence of fog and ice. Its season of navigation is shorter, and it has less rain, but has about the same snowfall and storm-wind velocities as the other Great Lakes. The seas, however, are more violent and the waves longer and higher on Lake Superior than on any other body of fresh water in America and probably in the world.

The prevailing storms are from the northeast, north and northwest, and during storms the water rises less on the lee shore and



LAKE SUPERIOR
FIG. 20.

falls less on the weather shore than on the shoaler lakes, as the greater depth facilitates the flow of the lower return currents.

Local magnetic attraction, or disturbance of the compass needle, is more prevalent than on the other lakes, and is strongest at particular localities along the north shore, a change of 19° occurring at one locality in a distance of but 650 ft. Fortunately, however, no disturbances of this magnitude have been encountered at any considerable distance from land.

The difference in level between Lake Superior and Lake Huron is overcome by canals and locks. There are two canals—one on the United States bank of the St. Mary's River, 1.6 miles in length, provided with two parallel locks of a single lift each, and the other on the Canadian bank of the river, 1.1 miles in length, provided with one lock of a single lift.

The larger of the United States locks, called the "Poe Lock," is 800 ft. long in chamber, 100 ft. wide, and has an available depth at average low-water stage of 20 ft. It has the distinction of being the largest canal lock in the world.

The smaller lock, the "Weitzel Lock," is 515 ft. long in chamber, 80 ft. wide, narrowing to 60 ft. at the gates, and has an available depth, at average low-water stage, of 14.5 ft. Both of the United States locks are operated by hydraulic power.

The Canadian lock is 900 ft. long, 60 ft. wide, with an available low-water depth of 20 ft. 3 in. This lock is operated by electricity.

Through the United States canal alone passes a marine tonnage much greater than that through any other canal in the world, the freight for 1902 aggregating 31 232 795 tons (of 2 000 lb.).*

Forces to Which Works on Lake Superior are Exposed, *i. e.*, Those Due to the Action of Wind, Waves, Seiches, Currents and Ice.

Wind.—The maximum observed wind velocities† on Lake Superior during the navigation season are given in Table 17.

In none of the places in Table 17, however, was the anemometer located where it could receive the full force of the wind

* All statistics relating to Lake commerce passing through the canals at Sault Ste. Marie have been furnished through the courtesy of the various officers of the U. S. Corps of Engineers, who have been in charge of the U. S. canal in recent years.

† As given by the U. S. Weather Bureau Meteorological Chart of the Great Lakes, No. 1, 1903.

which sweeps unobstructed over the open lake. This was clearly shown at Duluth, Minn., where the instrument was mounted above the roof of the Custom House, a building in the middle of the town, and near the foot of the plateau (which is about 635 ft. above lake level) which borders the north shore of the lake in this vicinity. In 1902 another anemometer was mounted on a structure just below the summit of the plateau, about 634 ft. above the lake, and about 419 ft. above the instrument previously described.

TABLE 17.

Months.	Wind velocity. Miles per hour.		
	Duluth.	Marquette.	Sault Ste. Marie.
April	58	49	60
May	60	52	50
June	48	45	45
July	48	68	42
August	50	48	50
September	78	61	45
October	55	46	50
November	56	48	52
December	60	54	54

The higher of the two instruments during storms showed wind velocities which averaged about 50% greater than those at the lower station, upon which official records had previously been based.

It is believed that the wind velocities shown at the upper station more nearly correspond to those experienced out on the open lake than do those indicated at the lower station, which must necessarily be affected by surrounding buildings and by the steep slope immediately in the rear. It is probable, therefore, that the velocities given in Table 17 are in most cases considerably smaller than those experienced on the open lake.

As far as works of harbor improvement alone are concerned, the action of the wind is harmful only in causing waves, currents, ice movement and changes of water level.

Waves.—The most dangerous forces to which marine structures on Lake Superior are subjected are those developed by wave action.

During the seasons of 1901-03, the writer secured more than 2,000 observations upon shallow-water waves at five different localities on the lake.

The observations were taken with a view of determining wave height, length and velocity; the relation of the wave hollow and crest to the undisturbed water level; the depth in which waves break; the effect of decreasing depth upon wave velocity; the relation between wind velocity and wave velocity; the static pressure due to passing waves; the maximum wave force developed, as measured by spring and by diaphragm dynamometers; the character of wave impact; the height of waves due to "fetch," and the reduction of wave height on entering a closed harbor.*

The greatest observed storm waves ranged in height (hollow to crest) from 14 to 23 ft., and in length from 200 to 275 ft., with velocities of propagation of from 25 to 33 ft. per second.

All observations upon storm waves were necessarily made in relatively shallow water, and no accurate measurements were attempted in the deeper waters more remote from shore. From a thorough investigation of the subject, however, it seems probable that during the most severe storms on Lake Superior, which occur only at intervals of several years, deep-water waves may be encountered 20 to 25 ft. in height and from 275 to 325 ft. in length.

The maximum readings obtained by spring dynamometers, during the seasons of 1901-02, at Duluth, Upper Entrance, Portage Canals, and Marquette, correspond to static pressures of 2 370, 2 525 and 2 055 lb. per sq. ft., respectively. As these readings extend through but two seasons, it is not probable that they indicate the maximum wave effect which may be exerted against works in their vicinity.

Seiches.—Even on a calm day the surface of Lake Superior undergoes continual fluctuations of level. These fluctuations, so noticeable upon the Great Lakes, when of considerable magnitude, are known as "seiches," and are usually formed as follows: When a storm wind blows in one direction for some time, its effect is to lower the water along one shore of the lake and raise it at the opposite shore, thus disturbing the normal condition of hydrostatic equilibrium. As the wind decreases, the water tends to regain a condition of stable equilibrium by a series of oscillations about a "nodal line" at right angles to the direction of the wind and about

* The discussion of these observations and of the results deduced has been printed as "Professional Papers, No. 31, Corps of Engineers, U. S. Army."

midway between the two shores. These oscillations at times continue for three or four days.

Many fluctuations in water level upon Lake Superior cannot be connected with known wind action, and their origin is attributed to rapid local changes in atmospheric pressure.

Seiches upon this lake seldom have a greater range than 3 or 4 ft., and this difference of level has been known to occur on some occasions in from 10 to 15 minutes. Fluctuations in level of from 6 in. to 1.0 ft. occur very frequently, and often this change of level is effected within a few minutes.

Owing to its greater depth, the fluctuations on Lake Superior are not as great as those upon a shallower lake, like Lake Erie, where, in November, 1901, the simultaneous difference of level between the two ends of the lake was 13.1 ft.

Currents.—The sudden fluctuations in lake level are of benefit in preventing or delaying the formation of ice at the entrances of harbors like Duluth-Superior, Grand Marais, Mich., and Upper Entrance, Portage Canals, which are provided with large interior basins; for as the lake rises or falls, a strong inward or outward current flows through the entrance channel, tending to prevent ice formation and causing a circulation of the water in the interior basin, which would otherwise become more or less stagnant and impure.

Sometimes the currents thus developed are so great as to threaten underscour, but it is generally easy to prevent this by revetting with rip-rap.

Lake Superior is so deep that littoral currents due to winds and waves are seldom strong enough to affect works constructed for harbor improvement.

Only two of the harbors on the lake, Duluth-Superior and Ontonagon, have rivers of any size emptying into them. At the latter place, owing to the small interior basin, the currents due to the river predominate over those due to fluctuations of water level, while at the former place the reverse is the case, except on those rare occasions when the St. Louis River is at an unusually high flood stage.

Ice.—On the harbors of Lake Superior, ice forms earlier, attains greater thickness, and melts later than on any other of the Great Lakes.

Dredging operations and navigation by small vessels are usually suspended on account of ice about the middle or latter part of November, but navigation by large vessels, assisted by ice-breaking tugs, is often maintained until about the 10th or 15th of December. Ordinarily, the lake proper does not freeze until early in February, although temperatures as low as 25 to 30° fahr., below zero, may have been experienced as early as the previous December.

At most of the harbors on Lake Superior, navigation is opened during the latter part of April.

Although the ice within the harbors varies in thickness from about 1 ft. to 3 or 4 ft., and on the lake from 1 ft. to about 2.5 ft., and, although ice-fields, miles in area, are moved by winds and currents against them, yet in no case has serious damage been inflicted by moving ice upon harbor works on Lake Superior.

These works are often severely tried by another phase of ice action caused as follows: Before the lake freezes over solidly, the water is often filled with floating pieces of ice. Should a storm then arise and the temperature be low, the water, spray and pieces of ice thrown by waves upon the breakwaters or piers are frozen together forming huge masses of solid ice upon the structures, and imposing heavy additional eccentric loads. For example, in the month of March, 1902, the west breakwater at the upper entrance, Portage Canals, was covered by a solid mass of ice 2 000 ft. long, 30 to 40 ft. wide, and 20 to 25 ft. high, which imposed an extra load of about 20 tons per lin. ft. upon the pile foundation. By April 19th, 1902, as the result of an unusually mild spring and of the wash of waves against the side next the lake, this mass of ice had been so melted away along that side, that the outer half of the deck of the breakwater was entirely free from ice, while the inner half still carried a heavy load, a large part of which actually hung over the interior face of the breakwater, and imposed a highly eccentric additional load upon the structure.

No injury of consequence has as yet resulted to harbor works on Lake Superior from the ice formation just described.

When the ice begins to move in the spring, "ice jams" in the entrance channels often occur. Due to this cause and to the action of masses of ice moved backward and forward by the currents, piers and breakwaters of timber, when not protected by steel or iron plates, are frequently chafed and worn near the level of the water

surface, and especially is this the case at the outer ends of these structures.

Ice action on Lake Superior at times serves to protect works designed for harbor improvement, for example, when the lake freezes over solidly in the vicinity of the works, wave action is entirely prevented for an average period of more than two months.

For some time before the lake freezes, and for a considerable time after the ice disappears from the lake, complete protection is afforded to the wooden decks of stone-filled timber breakwaters, and the separate cribs themselves are strongly united by a thick covering of solid ice formed by the freezing of water and spray thrown upon them by waves. This water is usually heavily charged with sand stirred up by the waves, which tends to increase the strength of the frozen mass. This was shown by tests for tensile strength made at the laboratory of the U. S. Engineer Office at Duluth, Minn., upon briquettes of standard form, both of clear ice and of wetted sand, frozen. The tensile strength of the former averaged about 312 lb. per sq. in., while the latter varied from 459 to 1 004 lb. per sq. in.

This result, which at first seemed paradoxical, was due to the fact that the briquettes of clear ice were formed by water frozen in place in the metal moulds, consequently the planes of crystallization were at right angles to the direction of the force, and the ice had its least strength; while in the case of the wetted and frozen sand, the individual grains of the latter so influenced the formation of the ice crystals surrounding them that the frozen mass was practically of uniform tensile strength in all directions.

To determine the effect of fineness of material upon the tensile strength of frozen briquettes of standard form, the following tests were made: Briquettes, of the proportions shown in Table 18, were made in a room of temperature 70° fahr., the temperature of the water, cement and sand being about 65° fahr. As soon as made, the briquettes were removed and exposed to a temperature of from 9 to 14° fahr., below zero, and were broken at the end of two hours.

Observations made from time to time showed that within a period of 24 hours ice of a greater thickness than 6 in. was never formed on still water at any of the United States harbors on Lake Superior, even with a temperature as low as 30° fahr., below zero.

TABLE 18.

No. of briquette.	Proportions (by weight).	Tensile strength per square inch.	Remarks.
1	150 cement; 25 water.	1 000+	Common sand was coarse, sharp grained sand from Superior Entry, passing a No. 10 sieve.
2	40 cement; 120 common sand; 13 water.	1 000+	
3	150 ground sand; 25 water.	1 004	Ground sand was same sand pulverized in a mortar so as to pass a No. 100 sieve, *i. e.*, of same fineness as the cement.
4	40 common sand; 120 ground sand; 13 water.	967	
5	Common sand, water completely filling voids.	459	

Works of Harbor Improvement, How Carried on.—Works of river and harbor improvement in the United States are planned, executed, operated and maintained under the direction of officers of the Corps of Engineers, U. S. Army, most ably and efficiently aided by a body of trained civilian assistants, many of whom are civil engineers of great experience and ability.

Authority for the commencement of a work must be given by Congress, and appropriations wherewith to prosecute the improvement come from the same source.

All plans and projects for commencing new works must be submitted by the U. S. Engineer Officer in local charge of the district, to the Secretary of War, through the Division Engineer and the Chief of Engineers, U. S. Army, for consideration.

Harbors on Lake Superior.

Compared with the other Great Lakes, Lake Superior is fairly well provided with natural harbors, but these are so situated that at the present time not one of them supports a marine commerce comparable with that of five or six of the more important artificial or semi-artificial harbors on the lake. In other words, the location of natural harbors did not serve to fix the lines of commerce, but, on the contrary, the requirements of commerce demanded the construction of harbors at other points on the lake.

Classification.—The harbors on Lake Superior may be divided into four general classes:

I. Natural harbors requiring little or no improvement, such as Washington and Rock Harbors, Isle Royale, and the harbors at Munising and Pequaming, Mich.

II. Harbors formed by connecting an interior basin or waterway with deep water in the lake by a channel dredged across a bar or spit and protected by parallel piers.

Duluth-Superior Harbor, and the harbors at Ontonagon and Grand Marais, Mich., are examples of this type.

III. Indentations or bays of considerable depth, with wide mouths opening lakeward, which natural openings have been partially closed by one or more breakwaters. Examples of this class are the harbors at Marquette and Presque Isle Point, Mich., Ashland, Wis., and Agate Bay, Minn.

IV. Protected basins entirely artificial. The only example of this type is found at the Upper Entrance of the Portage Canals.

Portage Entry, Portage Canals, is an example of a dredged channel maintained by a single structure fulfilling the double functions of a breakwater and of a pier.

The words "pier" and "breakwater," although in some places used interchangeably, are not so used on Lake Superior, where "pier" is applied to a structure, which helps to confine the currents in an entrance channel, and which is usually normal to the direction of the wave crests, whereas, a "breakwater" is generally parallel to the crests of storm waves, and is constructed to resist their impact.

Class I.

Harbors of this class being natural harbors not requiring improvement, it is unnecessary to discuss them further.

Class II.—Duluth-Superior Harbor, Ontonagon Harbor, and Harbor at Grand Marais, Mich.

Duluth-Superior Harbor.—This, the largest and most important harbor on the Great Lakes, is described on page 279.

Ontonagon Harbor.—This harbor is formed by the mouth of the Ontonagon River, one of the largest streams emptying into Lake Superior (see Fig. 2, Plate XX, and Fig. 7, Plate XXII). The improvement of the harbor was commenced in 1867, the first year that harbor improvement was undertaken by the United States on Lake Superior. In October, 1867, the depth which had formerly been about 9 or 10 ft. was reduced by a severe storm to but 6 ft. The project provided for maintaining a depth of not less than 12 ft., by the construction of two stone-filled timber-crib piers, one on

each side of the mouth of the river, and by dredging the channel between the piers. The project was completed in 1889.

The east pier is 2 315 and the west pier 2 675 ft. long, and their average distance apart is 236 ft. In 1895, the west pier projected 1 625 ft. and the east pier 1 850 ft. beyond the shore line, which is variable on account of shifting sand.

The river, when in flood, brings down a considerable quantity of sediment, some of which is deposited between the piers. As a consequence, dredging has to be done at intervals of several years, for example, during the summer of 1894 the channel between the piers was dredged so as to leave a least depth of 13.5 ft. On July 3d, 1902, no dredging having been done in the meantime, the least depth was 12 ft. 10 in. This depth was not, as in 1894, along the middle of the channel, but on a line parallel to and near the west pier, rendering dredging again necessary in 1903.

The marine commerce of the harbor being small, amounting in 1903 to but 7 990 tons (of 2 000 lb.), valued at $523 870, it is not probable that operations for materially increasing the present depth will be undertaken in the near future.

This harbor is of interest as representing the earliest type of pier construction on Lake Superior and as affording the only instance on the lake of an entrance channel maintained almost entirely by the current of a stream, but little assisted by inflow or outflow from an interior basin.

Grand Marais Harbor, Mich.—This harbor is situated on the south shore of Lake Superior, and is the only harbor of any kind along this dangerous stretch of coast, 90 miles in extent, on which numerous wrecks have occurred.

Originally the harbor was a deep-water bay, well protected from storms, with an anchorage area of 240 acres having a depth of not less than 18 ft.

The natural entrance, which was very wide, was at the eastern end and was obstructed by a bar, with a least depth of about 9 ft.

The project for the improvement of the harbor was adopted in 1881 and has not yet been completed. This project, as modified, provided for making a new entrance (see Fig. 1, Plate XXI), by cutting through the narrow sand spit separating the bay from the lake, constructing two parallel stone-filled timber-crib piers 500 ft. apart, dredging an 18-ft. channel between the piers and closing the

PLATE XX. VOL. LIV. PART A.
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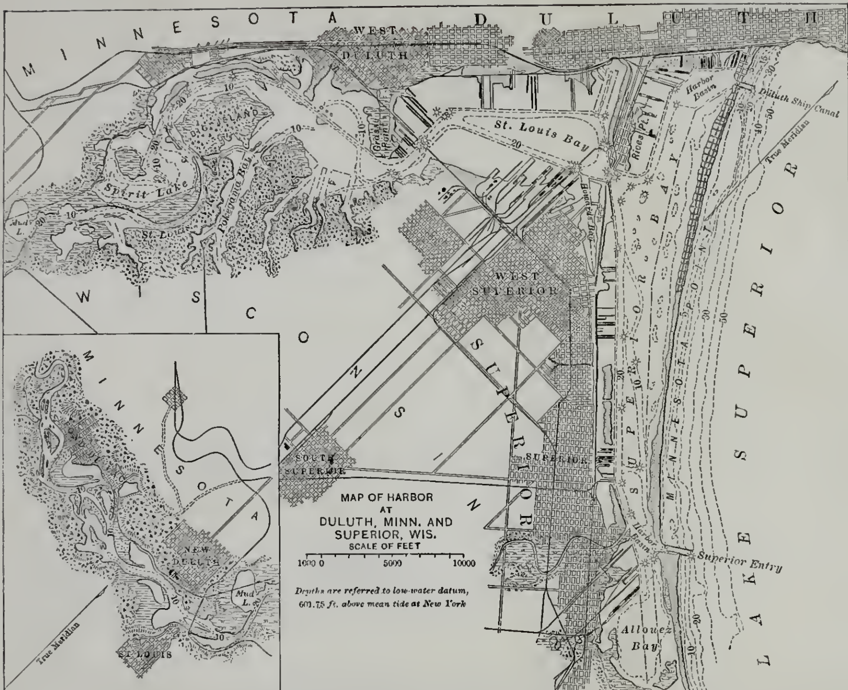
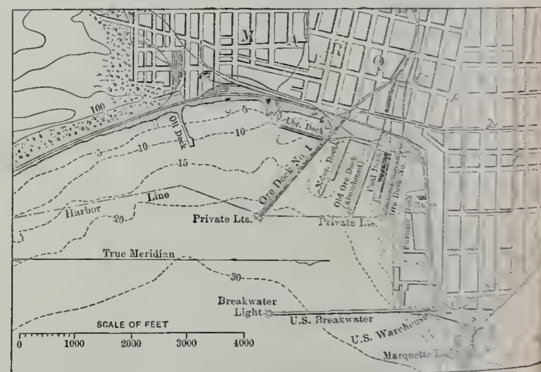
INTER. ENG. CONG., 1904.
GAILLARD ON
LAKE SUPERIOR HARBORS.



DULUTH-SUPERIOR HARBOR, MINN. & WIS.
FIG. 1

ONTONAGON HARBOR, MICH.
FIG. 2



MARQUETTE HARBOR, MICH.
FIG. 3

Figs. 1, 2, and 3, Plate 1, were prepared in 1903 under the direction of Major Lansing H. Beach, U.S. Corps of Engineers, by Mr. Wallace P. Welbanks, draftsman.

shallow natural entrance, 5 770 ft. in width, by a dike formed of a single row of strongly braced piles in contact, driven so as to present an inclined surface to the waves (see Fig. 4, Plate XXII). This dike, which cost but $4.72 per lin. ft., has withstood well the attack of waves and ice, and is building up a sand spit in its rear, which may in time close the original entrance completely.

The works at this harbor are more subject to the action of drifting sand than are those at any other place on Lake Superior. To such an extent had this action progressed prior to the commencement of improvements by the United States, that a large part of the harbor (East Bay) had been completely cut off by a sand spit which had formed on the east side of the natural channel.

From the commencement of the construction of the west pier in 1883 to July, 1902, about 1 260 000 cu. yd. of drifting sand had been caught and retained by that pier, causing the shore line at the pier to advance 700 ft. lakeward. This action will continue and in time will necessitate the extension of at least one of the piers.

On May 1st, 1904, the east pier had a total length of 1 545 ft., and the west pier 1 912 ft. (For cross-section of the piers, see Fig. 3, Plate XXII). The least depth in the entrance channel in December, 1903, was 13.0 ft.

The marine commerce of this harbor in 1903 aggregated 81 000 tons (of 2 000 lb.) valued at $1 624 824.

Class III.—Harbors at Marquette and Presque Isle Point., Mich., Ashland, Wis., and Agate Bay, Minn.

Marquette Harbor.—This harbor originally provided no shelter to vessels from easterly or northeasterly storms. In 1867 its improvement was commenced by the United States under a project providing for the construction of a stone-filled timber-crib breakwater, completed in 1894, which extends due south 3 000 ft. from the point of land east of the city and is the oldest breakwater on the lake. The stone-filled timber superstructure of this breakwater required extensive repairs, and consequently in 1895, the work of replacing it by one of concrete was commenced, and is still in progress, but it is expected that the work will be completed during the present season. Cross-sections of this breakwater are shown on Fig. 5, Plate XXII.

The area protected, wholly or in part, by the breakwater (see Fig. 3, Plate XX), is about 200 acres. This is ample for the needs of the commerce of Marquette, but owing to the large number of vessels which at times seek refuge there from storms, it is probable that the present breakwater will have to be extended in the near future.

The marine commerce of this harbor for 1903 aggregated 1 417 135 tons (of 2 000 lb.) valued at $6 317 186.

Harbor at Presque Isle Point.—This harbor is situated in an indentation of the shore only 2.5 miles north of Marquette Harbor. In its original condition the harbor was protected from northerly, and to some extent, from northeasterly storms (see Fig. 3, Plate XXI).

Further protection was afforded artificially by a stone-filled timber-crib breakwater, commenced in 1897 and completed in 1900. This breakwater was 1 050 ft. long and began at a point 120 ft. out from shore. It was constructed on the reef which runs out from the shore in a general southeasterly direction. During the season of 1902, the gap at the shore end of the breakwater, which had been widened by the action of waves, ice and currents, was closed by stone-filled timber-crib work, making the present length of the breakwater 1 266 ft. The total cost of the completed breakwater was about $54 400.

This breakwater is noteworthy, on account of its sloping superstructure (see Fig. 6, Plate XXII), its small cost, and the relatively large tonnage which has been developed, considering the amount expended on its construction.

The marine commerce of the harbor for 1903 aggregated 1 261 374 tons (of 2 000 lb.), valued at $3 379 445.

Ashland Harbor.—Ashland Harbor, located at the head of Chequamegon Bay, originally had no protection from swells which rolled into the bay, nor from waves generated within the bay itself (see Fig. 5, Plate XXI). The original project provided for constructing a single breakwater, 8 000 ft. in length, and for dredging a channel, 20 ft. in depth, in front of the wharves of the city. From motives of economy and because wave action was not so severe at this locality as at other points on the lake, it was decided to build this breakwater by driving a double row of piles at the proper dis-

PLATE XXI. VOL. LIV. PART A.
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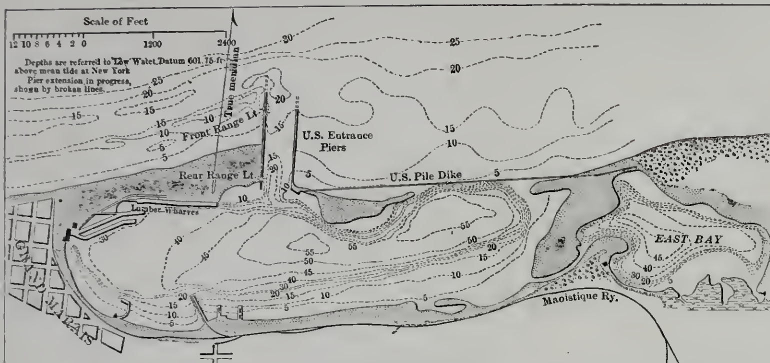
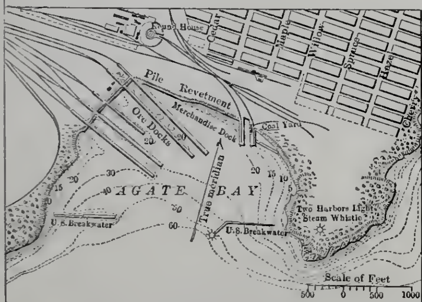


GRAND MARAIS HARBOR, MICH.

FIG. 1

ASHLAND HARBOR, WIS.

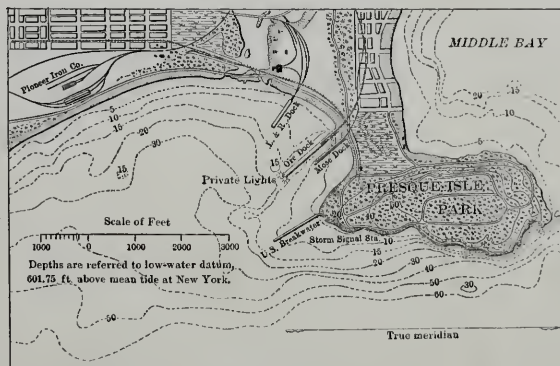
FIG. 5



AGATE BAY, MINN.

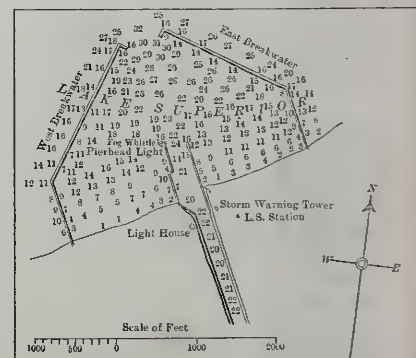
FIG. 2

Figs. 1,2,3 and 5, Plate II, were prepared in 1903 under the direction of Major Lansing H. Beach, U.S. Corps of Engineers, by Mr. Wallace P. Welbanks, draftsman.



PRESQUE ISLE HARBOR, MICH.

FIG. 3



UPPER ENTRANCE PORTAGE CANALS, MICH.

FIG. 4

tance apart, filling in the space between the piles with saw-mill "slabs," up to the level of the water surface, and weighting the "slabs" with heavy rip-rap (see Fig. 9, Plate XXII).

The main breakwater is 7 363 ft. long, and there are two detached shore spurs, 91 and 842 ft. long, respectively, the former being on the line of the main breakwater, and the latter, the result of subsequent modification of the original project, 2 600 ft. east of it.

This type of construction has nothing but its original cheapness, about $17.25 per lin. ft., to recommend it, the cost of repairs being excessive. To obviate future repairs, the main breakwater is now being utilized to form the hearting of a rubble mound, which is formed by piling heavy pieces of rip-rap along the sides and on top of the old breakwater (see Fig. 9, Plate XXII).

The work done at this harbor gives a protected area of about 1 600 acres, and affords safe anchorage and dockage for a distance of about three miles along the water front.

The vessel freight of the harbor for 1903 was 4 212 386 tons (of 2 000 lb.), valued at $20 904 356.

Harbor at Agate Bay.—Agate Bay, situated on the north shore of Lake Superior, 27 miles northeast of Duluth-Superior Harbor, is an important ore-shipping point for the iron mines of the Vermilion and Mesaba Ranges, and it is also used as a harbor of refuge.

It was originally a natural harbor with deep water at the entrance, and throughout much of its extent, but was directly exposed to winds from the southeast, through south, to S. S. W. and to heavy swells from northeast and southwest winds.

Operations for the improvement of the harbor were commenced in 1887 under a project providing for the construction of two breakwaters, one on each side of the bay, with a clear opening of 1 300 ft. between their outer ends, and a gap of about 100 ft. between the inner end of each breakwater and the rocky shore adjacent (see Fig. 2, Plate XXI). The east breakwater extends about S. 75° W. for 750 ft., and thence S. 30° W. for 310 ft.

The west breakwater extends about N. 75° E. for 900 ft. Both breakwaters are built of stone-filled timber cribs, 24 ft. wide, resting on an embankment of rip-rap, the top of which is 18 ft. below low-water datum. The deck of the superstructure is 6 ft. above the

same datum plane. A cross-section of the outer end of the east breakwater is shown by Fig. 10, Plate XXII. The breakwaters were completed in November, 1901, at a cost of about $234 000.

The area within the harbor, having a depth of not less than 18 ft., is but 70 acres. The slips between the ore docks occupy a considerable part of this area, leaving barely enough room inside for maneuvering vessels entering and leaving the slips. In consequence, the breakwaters have been repeatedly struck by vessels, and in one or two instances considerably damaged.

The east breakwater terminates in water 55 ft. deep, the greatest depth in which any construction of this character has been attempted on the Great Lakes. The vessel freight of this harbor has increased from 264 320 tons (of 2 000 lb.), valued at $524 800, in 1885, to 6 563 456 tons, valued at $15 789 960 in 1902.

During the same year, 3 662 vessels, of an average net registered tonnage of 3 000 tons each, arrived and departed at Agate Bay.

Class IV.

Upper Entrance. Portage Canals, Michigan.—The Portage Lake and Lake Superior Canals, Michigan, provide a waterway about 25 miles in length, crossing Keweenaw Point in a northwest direction from Keweenaw Bay to Lake Superior. It includes 5 miles in Portage River and its four dredged cuts, 17.5 miles in Portage Lake and 2.25 miles in the Upper Canal from the head of Portage Lake to Lake Superior, excavated through what was formerly dry land. The waterway was constructed by a private corporation, and was purchased by the United States in 1891, at which time it had a channel depth of but 12.5 ft.

The heavy tonnage charge upon commerce was abolished when the Government assumed control.

This waterway furnishes a shorter route for vessels bound up and down the lake, affords refuge from storms, and has assisted materially in developing the rich copper-mining region in the vicinity.

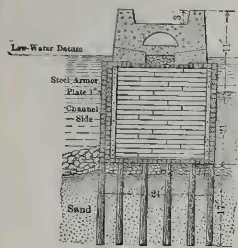
The approved project provides for a 20-ft. channel, with a least width of 120 ft., throughout the waterway.

At the entrance to the Upper Canal storms are more severe and frequent than at any other point under improvement on Lake

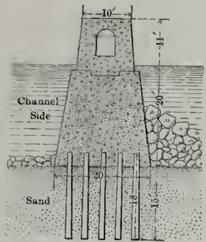
CROSS-SECTIONS OF VARIOUS WORKS
OF
HARBOR IMPROVEMENTS ON LAKE SUPERIOR

Prepared under the direction of
Captain CHARLES L. POTTER, U.S. Corps of Engineers
Duluth, Minnesota, June, 1904.

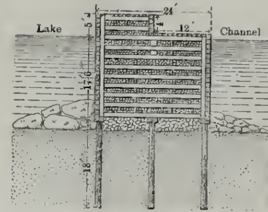
Scale of Feet
10 5 0 5 10 20 30 40 50



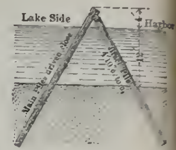
DULUTH SHIP CANAL PIERS.
Main Section.
FIG. 1.



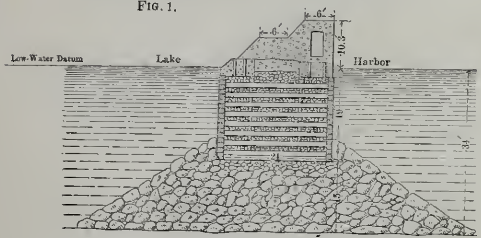
SUPERIOR ENTRY PIER.
200 feet from outer end.
FIG. 2.



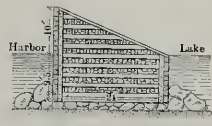
GRAND MARAIS, MICH., PIERS,
As extended in 1903
FIG. 3.



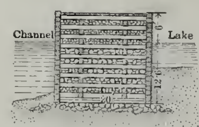
PILE DIKE
GRAND MARAIS, MICH.,
FIG. 4.



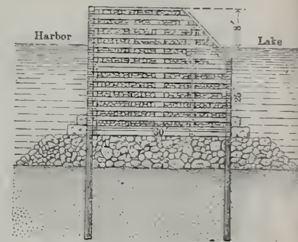
MARQUETTE BREAKWATER.
With new concrete superstructure.
FIG. 5.



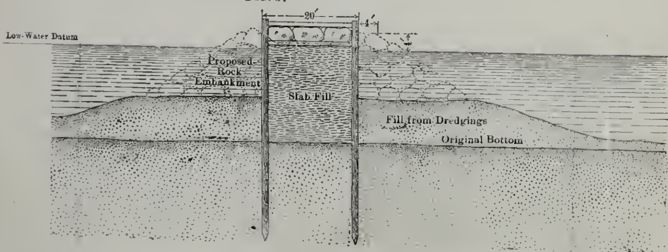
PRESQUE ISLE BREAKWATER
FIG. 6.



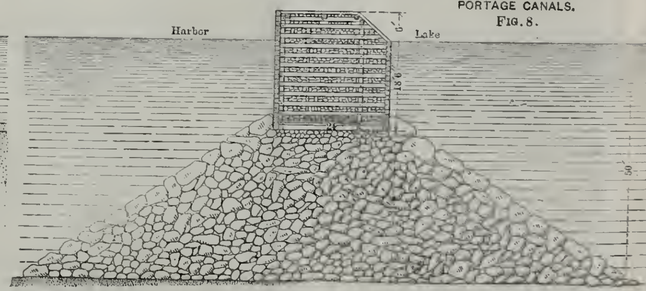
ONTONAGON ENTRANCE PIERS.
FIG. 7.



BREAKWATER, UPPER ENTRANCE,
PORTAGE CANALS.
FIG. 8.



ASHLAND BREAKWATER.
A rock embankment for strengthening the original structure
is now being built as indicated by dotted lines,
FIG. 9.



AGATE BAY BREAKWATER.
Section near outer end of the east breakwater.
FIG. 10.

Superior; consequently, it was decided to form an artificial harbor there which would afford refuge during storms and would facilitate the entrance of vessels into the canal in rough weather.

This was accomplished by the construction of two stone-filled timber-crib breakwaters commenced in 1898, and completed in October, 1901. Each is about 2 700 ft. in length. They start at the shore line about 3 000 ft. apart and extend out at right angles to its general direction for about 800 ft.

Each is then deflected about 45° toward the other and extends in the new direction about 1 900 ft., terminating in water 31 ft. deep, and leaving a clear opening 400 ft. wide at their outer ends (see Fig. 4, Plate XXI) through which vessels enter. This opening is 2 000 ft. from shore. The partially sheltered area of 106 acres inclosed by the breakwaters has a depth varying from shallow water alongshore to 31 ft. at the entrance. About 39 acres of this area has a least depth of 18 ft., and affords anchorage with fair shelter.

The average cost of the breakwaters proper, per linear foot, was $73.39. The cross-section is shown on Fig. 8, Plate XXII.

In 1903, there passed into this waterway 4 434 vessels, of an aggregate net registered tonnage of 2 416 729 tons, carrying 2 420 848 tons (of 2 000 lb.) of cargo, valued at $65 183 541.

Duluth-Superior Harbor, Minnesota and Wisconsin.

Duluth-Superior Harbor.—Situated at the head of Lake Superior, this is the most important harbor on the Great Lakes, on account of its size, the magnitude of its marine commerce, and its facilities for handling vessel freight. The established dock lines of the harbor have a frontage of 49 miles in all.

In 1903, but one harbor (New York) in the entire United States and all of its detached possessions equaled Duluth-Superior Harbor in the volume of its monthly marine commerce.

The harbor embraces the Duluth Canal, Superior Entry, Superior Bay, Allouez Bay, St. Louis Bay and St. Louis River to the limits of the Cities of Duluth and Superior, about twenty miles from Superior Entry (see Fig. 1, Plate XX).* The natural entrance from Lake Superior to Superior Bay, now known as Superior Entry, was

* A relief model of the harbor was shown in the Minnesota Exhibit, Mines and Metallurgy Building, Louisiana Purchase Exposition.

a winding channel, difficult to follow, over a shifting sand bar, and had an available depth varying from 9 to 11 ft.

The bays, before improvement, were broad expanses of shallow water about 8 or 9 ft. in depth, except along the channel through them, where the depth was greater but variable. The interior basin is separated from the lake by a remarkable "spit," of wave formation, 9 miles long and of an average width of about 200 yd., which forms a perfect natural breakwater.

Historical.—The United States commenced the improvement of the harbor, at Superior Entry, in 1867, under a project providing for building two parallel stone-filled timber-crib piers across the bar, 350 ft. apart, and dredging a 12-ft. channel between them. These piers still remain, but are to be replaced by new ones built entirely of concrete.

Operations were begun at Duluth in 1871 under a project providing for the extension of the stone-filled timber breakwater, commenced by the Northern Pacific Railroad, out in the lake, parallel to and about 2 000 ft. east of the northern end of Minnesota Point. The extension was completed for a distance of about 1 200 ft. from shore, but the superstructure and the upper part of the cribs were swept off by storms, leaving the remaining part of the breakwater submerged to a depth of from 1 to 8 ft. below low-water datum.

The Duluth Canal is an artificial channel cut through Minnesota Point by the City of Duluth in 1870-71.

In 1873, its maintenance and improvement were undertaken by the United States, in order to secure easy access to an inner harbor which would take the place of the exterior harbor previously described.

The improvements undertaken by the United States at this period afforded a navigable depth of about 12 ft., which was deemed sufficient for the needs of commerce at the time. The increase of commerce, however, soon necessitated further improvement of the harbor. This was authorized by the Act of Congress approved March 3d, 1881, and a new project was prepared which provided for securing 16-ft. navigation throughout the harbor and its two entrances. This project was completed in July, 1897, the desired depth having been obtained. The width of the dredged channels varied from 85 to 300 ft.

But the needs of commerce demanded further improvements, which were authorized by the Act of June 3d, 1896, and by modifications of August 14th, 1896, May 9th, 1901, and June 13th, 1902, providing for the widening and deepening to a navigable depth of 20 ft. of the existing channels, for new channels in Allouez Bay and St. Louis River, for extensive turning and anchorage basins of a navigable depth of 20 ft. at the junctions of two or more channels, for widening the Duluth Canal, and for rebuilding the piers at the Duluth Canal and at Superior Entry.

Dredging Operations under Present Project.—The project, before modification, provided only for deepening the channels and basins by dredging. The estimated amount of material to be removed was 20 870 355 cu. yd., and the estimated cost of doing the work was $3 130 553.

Keen competition marked the bidding on this dredging in February, 1897, and the lowest bids, 7.5, 8 and 10 cents per cu. yd., depending upon the locality, were very advantageous to the United States, the estimated cost having been 15 cents per cu. yd.

This work, which then involved the largest amount of dredging ever let at one time by the United States, was awarded to two firms, Williams, Green & Williams and Charles S. Barker.

Work under these contracts was commenced in June, 1897, and the contract with Charles S. Barker for the removal of 11 389 573 cu. yd. of material, was completed October 31st, 1902. That with Williams, Green & Williams, for 10 307 670 cu. yd., was completed November 14th, 1902, making a total of 21 697 243 cu. yd. of material dredged, more than half of which was paid for at 7.5 cents per cu. yd. The operations just described have given 17 miles of dredged channels from 120 to 600 ft. wide, and basins of an aggregate area of about 360 acres. The general depth is 22 ft., and no part of the dredged area has a less depth than 20 ft. at low-water datum.

The working season during the continuance of dredging operations averaged only about 7 months per annum. The combined plant engaged upon the work usually consisted of about 9 dipper-dredges, 1 hydraulic dredge, 13 tugs and 22 dump scows.

The dipper-dredges had buckets of from 3.5 to 8 cu. yd. capacity. The best monthly record made by any of the dipper-dredges was

122 713 cu. yd., and the best record per day (of 16 hours) was 7 126 cu. yd. The hydraulic dredge was provided with an 8-ft. centrifugal pump, 20-in. suction-pipe with rotary cutter, and 20-in. discharge pipe resting upon pontoons. Dredged material was at times discharged through as much as 2 000 ft. of pipe into shallow water, well removed from the channel.

The best monthly record made by the hydraulic dredge was 185 412 cu. yd., and the best record per day (of 24 hours) was 18 475 cu. yd. In all cases dredging was carried to a depth greater than 22 ft.

As this was one of the largest dredging contracts ever let in the United States, it may be of interest to state that during the seasons of 1899, 1900 and 1901, the time lost by the dredges operated by Williams, Green & Williams was 12.7, 14.4 and 26.0%, respectively, of the total time in which work was attempted. The corresponding figures for those operated by Chas. S. Barker were 24.3, 27.1 and 35.5%, respectively.

The loss of time was due principally to the necessity of repairs to machinery, but also to stormy weather, to waiting for tugs and scows, and to moving the dredges. As a rule, the entire dredging plant was repaired and put in good working condition during the closed season of navigation, and the repairs (mentioned in connection with the percentage of time lost) refer only to those necessitated by the continual breakage of machinery under the heavy strains to which it was exposed.

Surveys and Soundings through Ice.—To show the exact condition of the harbor at the close of each season's work, and to secure the data necessary for planning work for the next season, an accurate hydrographic survey of the entire harbor was made during the winter, experience having shown that from motives both of accuracy and economy it was better to take soundings through the ice in winter than from a boat in summer.

The positions of the proposed borings were laid out on the ice with accuracy and rapidity by means of a transit and steel tape.

The ice-boring machines were of the general type first used in 1892 on the Hay Lake survey, and described in the Annual Report of the Chief of Engineers, U. S. Army, 1893, p. 2964. An improve-

ment in the auger, devised in the U. S. Engineer Office, Duluth, Minn., increased the boring capacity of the machine, so that a 2.5-in. hole could be cut through solid ice 2 ft. thick in 5 seconds.

Between December 4th, 1902, and March 13th, 1903, 124 730 soundings were taken through ice varying in thickness from 4 to 48 in. and averaging 21 in., making the total thickness of ice cut through about 41.3 miles. Two parties were employed in doing the work, each equipped with an ice-boring machine and suitable sounding appliances.

Under favorable conditions a party could take 300 soundings, spaced 10 by 50 ft. through ice 2 ft. thick and water 23 ft. deep, in one hour. The best record for an 8-hour day was 2 749 soundings, through ice 13 in. thick and water 22 ft. deep. The cost of the soundings, for field work alone, was 3 cents each. Very little time was lost on account of weather, soundings being taken when the thermometer was as low as 30° fahr., below zero.

New Combined Timber and Concrete Piers, Duluth Canal.—The work of widening the entrance from 250 to 300 ft. and building new piers, at the Duluth Canal, was commenced in 1898 and completed in 1901 at a cost of over $650 000, including the price of land. The two piers, which are similar in dimensions, extend from the deep water of the inner harbor out to the 23-ft. depth in the lake.

The clear width between them for a distance of 1 250 ft. from the outer end is 300 ft.; after this, they flare out like the mouth of a trumpet, to a width of 540 ft., at the inner end. The length of each pier is about 1 700 ft., and it projects about 1 150 ft. beyond the shore line.

The stone-filled timber cribs forming the substructure are each 100 ft. long, 24 ft. wide and 21 ft. deep. Each crib rests upon 150 piles sawed off near the level of the bottom (see Fig. 1, Plate XXII). The cribs are armored for a width of 1 ft. on top of the berm and for the upper 7 ft. of the inner face, with steel plates 1 in. thick, as a protection against ice and the impact of vessels.

The concrete superstructure is built in monolithic sections 10 ft. long, and extends from 1 ft. below low-water datum to a height of 10 ft. above this plane, except near the outer end, where it reaches a height of 18 ft. above low-water datum. Each monolith of the

typical section contains 42.3 cu. yd., and weighs 85.6 tons. The largest monoliths at the pier heads, which are much wider and higher, contain 209 cu. yd. each, and weigh 423 tons.

During severe storms the wave crests overtop the parapet walls of the piers by several feet, and at such times it would be impossible for the keepers to reach the light at the outer end by walking on the pier itself. To overcome this difficulty a low gallery is made in the concrete superstructure, terminating in two arched chambers under the light-house. In this gallery runs a car propelled by a person reclining on his back and working pedals with his feet.

Owing to the compressibility and elasticity of the deep timber cribs upon which they rest, even the heaviest concrete monoliths during severe storms have been observed to have, at frequent intervals, a total lateral movement of as much as 0.1 in., due to the action of high waves traveling parallel to the piers.

Iron lamp-posts have been broken off 20.25 ft. above low water datum, both by waves and by blocks of ice hurled up by waves, but the piers themselves have not suffered the slightest injury either from waves or ice.

These piers were designed by Lieutenant-Colonel Clinton B. Sears, U. S. Corps of Engineers, ably assisted by Messrs. J. H. Darling and Clarence Coleman, U. S. Assistant Engineers, and are probably the finest examples of combined timber and concrete entrance piers on the Great Lakes.

*New All-Concrete Piers, Superior Entry.**—The original stone-filled, timber-crib piers at Superior Entry were placed about 33 years ago in water of an average depth of 8 to 10 ft., the object then being to secure a channel depth of 12 ft. With the growth of marine commerce, the depth in the entrance channel has been successively increased until at the present time it is 24 ft., and the crib bottoms are therefore from 14 to 16 ft. above the bottom of the channel. Owing to this, and other causes, considerable lateral displacement of the cribs resulted, and it was evident that the piers would soon have to be replaced by new ones. Accordingly, early in 1902, the writer, then in charge of harbor improvements on Lake Superior, submitted a project, approved February 6th, 1902,

* Models of these piers, and of those at the Duluth Canal, and a model of the breakwater at Marquette, Mich., were exhibited in the United States Government Building, in the exhibit of the Corps of Engineers, U. S. Army, Louisiana Purchase Exposition, where there was also a model of the concrete bucket and of the concrete moulds.

for replacing the old piers by new ones to be built entirely of concrete, in monolithic sections, 16 ft. long, moulded *in situ* around and upon foundation piles, sawed off a short distance above the level of the bottom (see Fig. 2, Plate XXII).* In preparing the project and designing the plant for this, as yet untried class of pier work, the writer received invaluable and much appreciated assistance from Mr. Clarence Coleman, U. S. Assistant Engineer, and Mr. M. W. Lewis, U. S. Junior Engineer.

In constructing the piers, concrete is deposited in still water within the mould from 2-yd. buckets, each provided with two leaves opening outward from the bottom and with two weighted canvas protection flaps over the top.

The bucket is filled with concrete, lowered slowly and gently into place, the catch closing the bottom doors is opened, and the bucket slowly raised up from the concrete, leaving the latter in place and undisturbed by "wash," as is shown by the fact that there is but little discoloration of the water within the mould even after 75 or 100 cu. yd. of concrete have been deposited within it.

The moulds, which are of wood, and weigh, when loaded with ballast for submergence, about 40 tons, are assembled above water and lowered into place from within the interior of a 4-hoist "traveler," which runs upon a track of 31-ft. gauge, at a speed of about 90 ft. per min.

The moulds are taken apart under water, after the concrete has set, by means of rods with threaded ends, which are withdrawn from an eye in a tie-bar by a wrench operated above water.

All materials are handled by machinery, and the plant is so well adapted to the work in hand that although it has a capacity of from 250 to 300 cu. yd. of concrete in 8 hours, the total force engaged in handling materials, mixing and handling concrete, assembling and taking off moulds, operating machinery, etc., averages only about 50 persons.

The piers are to be parallel and 300 ft. apart for about 2 250 ft. from the outer ends. They then begin to flare out, so as to give a trumpet-shaped opening at the inner end. The south pier is to be 2 961 ft. long, and the north pier 3 418 ft. The estimated cost of the two piers, including the cost of removal of the old piers, is $925 000.

* The rip-rap shown on the right of the cross section is to be thrown there, when the old pier is removed, simply to get it out of the way.

If completed within the estimate, as seems probable, the cost per linear foot will be considerably less than for the piers at the Duluth Canal. The construction of the south pier was commenced in 1903, and it will probably be completed in the fall of 1905.*

Marine Commerce of Duluth-Superior Harbor.—The vessel freight received and shipped at Duluth-Superior Harbor has increased from 2 848 672 tons (of 2 000 lb.) in 1890, to 17 966 718 tons, valued at $177 594 212, in 1903. Among the items for 1903 are 52 415 500 bushels of barley, oats, rye, wheat, corn and flax; 4 219 211 tons of coal; 10 387 457 tons of iron ore, and 407 416 000 ft. B. M. of pine lumber.

The navigation season for 1903 lasted 245 days† and during this period, exclusive of tugs and scows, 10 119 vessels of an aggregate net registered tonnage of 16 515 867 tons entered and departed. The average net registered tonnage of these vessels (exclusive of tugs) was 1 841 tons in 1903, having gradually increased from 1 122 tons in 1895.

The vessel freight handled in this harbor in 8 months is so enormous that it is difficult for the mind to grasp its volume. Some appreciation of its magnitude may be obtained if we suppose it all to be loaded on ordinary freight cars, 40 000 lb. to the car, and the cars themselves to be placed on the track as closely as possible to one another. So arranged, the 900 000 cars required would occupy every foot of space on a double-track railroad extending from New York City to San Francisco.

Cost of Improvements, Duluth-Superior Harbor.—From the commencement of operations by the United States in 1867 to December 31st, 1903, $4 590 905.16 have been expended by the United States in the improvement of Duluth-Superior Harbor, and during the same period the vessel freight arrived and departed has aggregated about 138 930 884 tons (of 2 000 lb.), valued at $1 986 847 470.

Modifications in Methods, in Materials and in Cross-Section, on Lake Superior During the Past Decade.

Methods: Appropriations.—One of the most important and advantageous modifications of recent years on Lake Superior is that

*The work is being done under the direction of Capt. Chas. L. Potter, U. S. Corps of Engineers.

†This refers to interlake navigation. Navigation between Duluth-Superior Harbor and other Lake Superior ports lasted for 290 days.

relating to the method of making appropriations for the prosecution of works of harbor improvement.

Previous to 1896, appropriations for any particular work were ordinarily small in amount and made at irregular intervals. This increased the cost of work and added greatly to the difficulty of its execution, since under the "piece-meal" method of construction necessarily adopted, neither the officer in charge of the work nor the contractor was justified in going to much expense in providing special plant, while the work itself, being for years in an unfinished condition, often suffered injuries which would not have been experienced had funds been available with which to hasten its completion.

As an instance of a work carried on as just described, the improvement of the harbor at Grand Marais, Mich., may be cited. From June, 1880, to the present time, a period of more than 24 years, there have been twelve different appropriations for this harbor, aggregating $400 250. The piers are still uncompleted, although parts of the superstructure, built 15 or 18 years ago, have suffered greatly from decay.

In pleasing contrast is the greater liberality shown in recent years in appropriations. This change was inaugurated on Lake Superior in June, 1896, when an appropriation was made for the improvement of Duluth-Superior Harbor, and contracts were authorized for the entire improvement contemplated, estimated to cost $3 130 553. So reasonable were the proposals submitted, that the entire work contemplated was successfully accomplished in 5.5 years at a cost more than a million dollars under the original estimate. The same liberal and broad-minded policy has been pursued in providing appropriations of $1 065 000 for the improvement of the "Waterway from Keweenaw Bay to Lake Superior" (Portage Canals), and of nearly $100 000 for the completion of the concrete superstructure of the breakwater at Marquette, Mich.

Foundations.—Piers of the older type were constructed by excavating a trench, leveling its bottom as well as practicable, and sinking cribs on the bottom thus prepared.

As a result of unequal settlement, erosion of the bottom alongside the pier, or of removal of material from beneath the cribs by cross-currents, displacement of the cribs sometimes resulted, and

especially was this the case with works where the bottom was very sandy or where the original channel depth had to be increased to meet the growing demands of commerce.

To avoid this trouble, all piers planned or constructed on Lake Superior in recent years are supported upon a foundation of piles sawed off on, or near, the bottom.

This construction prevents lateral or vertical displacement of the cribs from the causes mentioned, and has proved preferable to the use of fascine mats, which were tried with only partial success at Grand Marais, Mich., and afterward abandoned in favor of a pile foundation.

Where rock is cheap, and timber becoming more expensive every year, motives of economy render it necessary that in water of considerable depth the cribs of breakwaters shall rest upon a mound of stone, flattened and leveled on top.

At present prices a stone-filled timber crib of the most modern type, including pile foundation, costs, in place, about $5 per cu. yd., while the stone embankment, in place, does not cost over $1.50 per cu. yd. It therefore results that when the depth of water is known, and the width of the crib has been established, the height of stone embankment, corresponding to the most economical cross-section of breakwater, can readily be computed.

To prevent unequal settlement, which is always likely to occur in breakwaters when the cribs rest upon a stone embankment of considerable height, the breakwaters at the Upper Entrance, Portage Canals, rest upon piles, sawed off, as shown in Fig. 8, Plate XXII, the stone embankment extending to the level of the top of the piles. The method of construction used in this case has proved very successful.

Cribs.—The modifications in timber cribs during the past decade have consisted in an increase of general strength and length; in decreasing the size of the timber ties, which were formerly unnecessarily large; in adjusting the crib length to conform to the merchantable length of timber as cut for the general market; in the general use of machinery for framing crib timbers, and in plating with steel, as a protection against ice, vessels and rafts, the upper part of the inner faces of cribs along entrance channels.

Pile and Slab Type of Breakwaters and Piers.—It is not prob-

able that a rock-weighted "pile and slab" type of breakwater will ever be used again on Lake Superior, as it is expensive to maintain and does not withstand well the attack of waves.

On account of the limited appropriation, and because the exposure is not great and the general direction of wave travel is parallel to the piers, modified rock-weighted "pile and slab" piers are being constructed at Port Wing, Wis., on the south shore of Lake Superior. These piers are planned so that all ties, waling-pieces and other strengthening timbers are below water level, and therefore not subject to decay. To give additional strength, heavy rip-rap is piled along the outer side of each pier.

Decking.—No part of the rock-filled breakwater is so severely tried by wave action as is the deck, and upon its integrity the safety of the entire structure may rest. When storm waves strike the vertical face of a breakwater, a mass of water is thrown upward to a considerable height and falls upon the deck of the structure. When, as in the older types of construction, the deck plank were but 3 in. thick, damage was at times inflicted by the mass of falling water, both upon the deck plank and upon the supporting timbers on which they rested.

Deck plank were at that time placed in contact with one another, and being alternately wet and dry, they suffered so from decay that renewal was necessary at intervals of from 8 to 12 years. In order to permit free circulation of air, deck plank are now laid with a space of about 1.5 in. between them and in consequence their decay has been considerably retarded.

To prevent damage from falling water, the thickness of the deck plank has been increased to 6 in., and the supporting timbers have been placed more closely together.

As a further and more important modification tending to reduce the strain upon the entire superstructure and especially upon the deck, the exposed face of every recent breakwater has been built with a slope, the effect of which, during storms, is to cause a portion of the wave energy to be expended in lifting a mass of water and projecting it in an inclined direction above and over the deck of the breakwater, practically no water falling normally upon the deck during storms.

Width of Entrance.—In the older works, the width of entrance

channels varied from 250 ft. at Duluth Canal, to 500 ft. at Grand Marais, Mich. In the former case, the width was scarcely sufficient to permit large steamers to pass one another readily, while in the latter case the entrance is so wide, as compared with the area of the interior basin, that the currents, which are developed solely by fluctuations in lake level, are not sufficient to render the channel self-sustaining. In fixing the width of entrance channels on Lake Superior there is invariably a conflict between two opposing considerations. For ease of ingress and egress of vessels, a wide channel is desirable. To reduce the disturbance caused by swells which enter the harbor, a channel should be as narrow as practicable.

After due consideration and as a compromise between these conflicting conditions, the width of channel at the Duluth Canal and at Superior Entry has been fixed at 300 ft. Storm waves and the winds which cause them move in a direction parallel to the piers, and consequently, although in 1903 over 9 800 vessels (including tugs and scows) passed through the Duluth Canal in about eight months, with a single exception, no difficulty was experienced either in entering or departing. Owing to the fact that the Duluth Canal was constructed so near the extreme north end of Superior Bay, the swells which enter the bay through this canal, have but little room toward the north in which to expand, and consequently cause some inconvenience in this vicinity for short periods during severe storms. This trouble would have been avoided had the City of Duluth cut the canal 3 000 or 4 000 ft. south of its present location.

Modification in Materials: Concrete.—Until within the past seven or eight years, timber and stone were the two materials of which all breakwaters and piers on Lake Superior were constructed. Their universal use arose from three reasons: (1) the abundance, (2) the cheapness of stone and timber, and (3) the ease with which timber cribs could be built in sheltered localities, partly weighted with ballast, and then towed to their proper places and sunk. White pine timber continually submerged in fresh water is practically imperishable, but if alternately wet and dry it decays in from 8 to 15 years.

Owing to the rapid exhaustion of the immense pine forests adjacent to the Great Lakes, it is now practically impossible to obtain native timber of unusual dimensions in the vicinity of Lake Su-

perior, and for the past two years it has been necessary to procure the larger pieces of timber from the Pacific Coast, over 2 000 miles distant.

As a consequence of its increasing scarcity, timber which sold at $11 per M. ft., B. M., in 1893, cost about $18 per M. ft. in 1903.

On the other hand, Portland cement has greatly decreased in price during the same period, and as sand, pebbles and broken stone are plentiful and their prices reasonable, it would appear to be but a question of time when concrete will supersede stone and timber in breakwater and pier construction. The time has already arrived, so far as concerns the superstructures of breakwaters and piers, which require renewals at intervals of 12 or 15 years at most. The original cost of the superstructure plus a single renewal now nearly equals the cost of a modern concrete superstructure. It is therefore probable that in the future, when renewal is required and adequate funds are available, the rock-filled timber superstructures of all breakwaters and piers will be replaced by concrete superstructures, as is now being done at Marquette, Mich.

Including the pile and stone foundation and steel armor, the cribs at the Duluth Canal cost about $5 per cu. yd. in place. The submerged weight of such a crib is about 50 lb. per cu. ft., while the submerged weight of concrete is 87 lb. per cu. ft. It is therefore evident that for equal stability against overturning or against displacement by waves, the area of cross-section of a concrete pier or breakwater would be considerably smaller than in the case of one of crib work. The foregoing considerations, confirmed by the results of a preliminary test, induced the writer early in 1902 to submit a plan and project for piers entirely of concrete for Superior Entry. The project was approved and the piers are now being constructed and are estimated to cost less per linear foot than if built of rock-filled timber cribs of equal stability surmounted by a concrete superstructure.

Pebbles and Broken Stone.—At certain localities on Lake Superior pebbles are very abundant and of excellent quality. The sizes are generally so nicely assorted by Nature that the percentage of voids is smaller than in the case of broken stone, and consequently less cement mortar is required to fill the voids.

At equal prices per unit for all materials, concrete of pebbles is therefore cheaper than that of broken stone, and as pebbles cost

less per cubic yard on Lake Superior than broken stone, the latter has been but little used on Government work in the vicinity of the lake. In the construction of the concrete superstructure of the Duluth Canal piers it was found that screened broken stone contained 47.5% of voids, while pebbles contained but 36.5 per cent. The use of pebbles instead of this broken stone here resulted in a saving of about $15 000 on cement and sand alone, the amount of concrete in the work being about 20 000 cu. yd.

Galleries.—The substitution of concrete for stone and timber in the superstructure of piers and breakwaters has permitted the easy construction of galleries within them, by means of which during storms, light-keepers residing on shore can reach in safety the lights or fog signals at the outer ends of these structures.

At the Duluth Canal the gallery is low, and the light-keeper reclines in a special car, propelled by movement of the feet. At Marquette and at Superior Entry, the gallery is high enough for a person to walk in, but it is proposed to use a car which the light-keeper may propel while sitting.

Effect of Harbor Improvements upon Transportation Methods, Tonnage and Freight Rates.

It is difficult to realize the enormous growth of the marine commerce of the Great Lakes during recent years, and especially remarkable has been the development on Lake Superior. In 1883 the total amount of freight passing through both the United States and the Canadian Canals at Sault Ste. Marie, which is an accurate measure of the volume of through-vessel freight on Lake Superior, aggregated 2 267 105 tons (of 2 000 lb.). In 1893 this freight had increased to 10 796 572 tons; while in 1903 it reached the enormous total of 34 674 437 tons, over 85% of which was *en route* to or from Lake Erie ports.

It will doubtless be a surprise to many to learn that during the period of ten years from 1894 to 1903, inclusive, the aggregate gross tonnage of the steam vessels of steel constructed on the Great Lakes was 31% greater than on the entire Atlantic and Pacific seaboard of the United States.

According to Lloyd's register for 1903-04 the gross tonnage of all sea-going steam merchant vessels of the United States of over 100 tons is 1 220 995 tons, and of the same class of vessels on the

Great Lakes, 1 001 072 tons. As the construction of steel steam vessels is proceeding more rapidly on the Great Lakes than on the seaboard, it would seem to be but a question of a few years before the aggregate tonnage of the United States vessels of this type on the Great Lakes will exceed that of the same type of U. S. vessels upon the sea.

The development of this immense lake commerce has been rendered possible by Government improvement of the harbors of the Great Lakes and of the waterways connecting them.

In 1883 the governing low-water depth in Duluth-Superior Harbor was but 12 ft., while at Sault Ste. Marie, it was 14.5 ft. At the present time the governing depth at five of the principal improved harbors on Lake Superior is 20 ft., and at five others it ranges from 12.5 to 18 ft. On the waterways connecting the Great Lakes the governing depth is nearly 20 ft.

The increase of depth obtained by means of Government improvements has had a marked effect upon transportation methods, and has resulted in a steady increase in the size of vessels and consequently of their carrying capacity. In 1883 the average cargo carried by vessels passing through the Sault Ste. Marie Canals was 473 tons (of 2 000 lb.); in 1893 it was 899 tons, and in 1903, 1 865 tons. The latest and largest type of Lake ore carrier is 560 ft. long, 56 ft. beam, 33 ft. deep, and is designed to carry 12 500 tons. The keel of this vessel, the *Augustus B. Wolvin*, was laid December 3d, 1903, and the vessel was delivered to the owners, completed and loaded with 10 300 tons of soft coal, June 7th, 1904—six months and four days after the first keel plate was laid.

With increase of depth and growth of commerce equal progress has been made in improving facilities for handling freight until at the present time vessels are loaded and unloaded on the Great Lakes with a rapidity unequalled at our ocean ports; for example, a cargo of 5 217 tons of iron ore was loaded on a vessel, in Duluth-Superior Harbor in a little less than 31 min., and unloaded at Conneaut, Ohio, in 3 hr. 55 min.

When the depths were small and the carrying capacity of vessels limited, vessel owners endeavored to compensate for limited carrying capacity by employing steamers to tow one or more barges or schooners. In 1870 the number of schooners, barges, etc., passing through the canals, at Sault Ste. Marie, was more than three times

greater than the number of steamers; in 1880 the two classes of vessels were equal in number, while in 1903 the steamers outnumbered the towed vessels nearly in the proportion of 4 to 1. There are now no sailing vessels, except small pleasure craft, on Lake Superior.

The present tendency of interlake transportation is unquestionably toward single carriers of large capacity and moderate speed, constructed with numerous hatches to facilitate the operation of loading and unloading.

Due to increased depth, to the resulting increase in the capacity of vessels and to improved appliances for handling freight, there has been a steady but somewhat irregular reduction in freight rates on the Great Lakes. In 1887 the average haul for all freight passing through the Sault Ste. Marie Canals was 811.4 miles, and the average freight charge per ton per mile was 2.3 mills. In 1903 the average haul was 835.6 miles and the average freight charge per ton per mile was 0.92 mills. Notwithstanding the low freight rate, freight charges amounting to $26 727 735 were paid in 1903 upon the total freight passing through the two canals at Sault Ste. Marie.

The movement of freight by vessels and estimated value of same, at the principal ports on Lake Superior, for 1903, and since the commencement of improvements by the United States, is shown in Table 19, which also contains the total cost of harbor improvements, maintenance and operations, at each of the localities mentioned, to January 1st, 1904.

Cost of Harbor Improvements on Lake Superior, and Financial Effects of Same.—It will be seen from Table 19 that up to January 1st, 1904, the sum of $8 132 822.98 had been spent upon the improvement, maintenance and operation of the United States harbors on Lake Superior, and that during the same period the vessel freight handled at these harbors aggregated 284 397 822 tons (of 2 000 lb.), valued at $3 241 139 028.

From these figures it can be shown that the total sum expended on the various harbors on Lake Superior since the commencement of improvements by the United States, if imposed as a tonnage tax would have amounted to 2.86 cents per ton of vessel freight and corresponds to about one-fourth of 1% of the money value of this freight.

TABLE 19.—TOTAL VESSEL FREIGHT AND ESTIMATED VALUE OF SAME, BOTH FOR CALENDAR YEAR ENDING DECEMBER 31ST, 1903, AND SINCE THE COMMENCEMENT OF OPERATIONS.

Lake Superior District (furnished through courtesy of Capt. Chas. L. Potter, U. S. Corps of Engineers, in charge of Lake Superior District).

Name of Work.	Vessel Freight. 1903.		Date of Commencement of Improvements by the United States.	Vessel Freight from Commencement of Work to Jan. 1st, 1904		Total amount expended on improvements operation and maintenance, from commencement of work to Jan. 1st, 1904.
	Tons of 2 000 lb.	Estimated Value.		Tons of 2 000 lb.	Estimated Value.	
Grand Marais, Minn.	35 255	$732 137	1880	115 147	$3 797 976	$164 908.59
Agate Bay, Minn.	5 986 759	15 042 922	1887	45 079 518	94 188 900	244 183.81
Duluth-Superior, Minn. and Wis.	a17 966 718	177 744 956	1867	138 930 884	1 986 847 470	1 590 905.16
Port Wing, Wis.	51 969	785 054	1903	124 119	1 743 454	21 378.57
Ashland, Wis.	b4 212 386	20 904 356	1887	51 830 948	454 538 922	800 692.96
Ontonagon, Mich.	c7 993	523 870	1867	2 718 851	71 889 521	349 555.57
Waterway across Keweenaw Point	de2 430 848	65 183 541	1891	16 355 768	513 474 260	1 295 000.00
Operating and care of above waterway	....	....	....	....	....	131 046.74
Marquette, Mich.	g2 678 509	69 696 631	1867	g28 169 285	h130 717 931	603 879.35
Harbor of Refuge, Presque Isle Pt.	(1 261 374)	(3 379 445)	1897	(10 115 990)	(25 005 816)	49 590.58
Harbor of Refuge, Grand Marais, Mich.	f81 000	1 624 824	1880	1 073 299	13 915 591	381 681.65
Totals	33 441 434	$292 238 291	....	284 397 822	$3 241 139 028	$8 132 822.98

a Does not include 81 125 000 ft., B. M. of logs. b 250 000 000 ft. c 2 400 000 ft. d 17 787 000 ft., a total of 351 312 000 ft. e Largely freight passing through canals. f Includes logs. g h Includes Harbor of Refuge, Presque Isle Pt., Mich.

The reduction in freight rate per ton mile on all Lake Superior freight since 1887 has been 1.38 mills, a reduction which, if applied to the volume of freight carried in United States vessels through the canals at Sault Ste. Marie in 1903, would amount to \$37 780 342, a saving in one year more than four and a half times greater than the cost of all United States improvements on Lake Superior from 1867 to January 1st, 1904.

As wages, fuel, oil and other operating expenses, are fully as high at the present time as in 1887, this decrease in freight rates must be due primarily to the following causes: (*a*) Government improvement of the rivers and harbors on the Great Lakes and their connecting waterways, and (*b*) improved facilities for loading, unloading and handling vessels.

While it cannot be stated definitely, exactly what proportion of this annual saving of \$37 780 342 in freight, as compared with freight charges in 1887, is attributable to river and harbor improvements on Lake Superior alone, yet it is safe to say that the amount thereby saved each year exceeds the aggregate cost of all river and harbor improvements on Lake Superior during the past 37 years.

The saving in vessel freight, however, represents but a small part of the benefit accruing from Government improvements on the Great Lakes. Water competition has cheapened freight rates by rail over thousands of miles of railroad; the development and growth of many cities, and of a huge territory have been greatly facilitated by lake commerce, and millions of dollars of capital have found profitable investment in vessels, docks, shipyards, elevators, warehouses, etc., along the shores of the lakes.

The value of the United States craft passing through the canals at Sault Ste. Marie has risen from \$2 089 400 in 1887 to \$68 252 800 in 1903, and the safety of this immense fleet and its cargo, valued at \$330 254 618, has been greatly enhanced by the number of excellent and well-lighted harbors which have been constructed by the United States on Lake Superior.

Taking into consideration all advantages resulting from the improvement of the rivers and harbors of the Great Lakes, and of their connecting waterways, it is doubtful whether any like sum expended by the Government during the same period has brought in quicker returns, or has more richly repaid the original investment.

TRANSACTIONS

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HARBORS.

SEACOAST HARBORS IN THE UNITED STATES.

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The essential differences between seacoast and interior harbors in the United States are due to three things, *viz.,* the tidal influences, great ocean currents, and exposure to storm action.

General Description.

The characteristics of our seacoast harbors vary with the locality. They may be roughly grouped as follows:

New England Harbors.—On the coast from Maine to Cape Cod and on the mainland from Cape Cod to New York, the harbors are mostly rock-bound glacial channels, sometimes obstructed by rock ledges or coarse glacial detritus. The tidal rise is moderate at New York and small at Cape Cod, increasing very much north of that point. Great ocean currents are of little importance here and, while storms are severe, the wave action does not produce many rapid changes, on account of the resistant nature of the materials. These harbors have generally been improved by dredging and subaqueous rock removal.

Between Cape Cod and New York there is a series of island harbors like Nantucket, Vineyard Haven and the harbors in Block

Island and Long Island that are somewhat peculiar. The islands are great terminal moraines, made up of all kinds of materials, from large boulders to fine sand. The harbors are formed by the irregularities in the contours of the old moraines, somewhat modified by wave and current action. The tidal rise here is only a few feet but, on account of great interior sounds and bays to be filled, the tidal currents are, in some places, very strong. Storms are severe and the wave action on the finer materials quite considerable. These harbors have been improved both by dredging and by works for contraction and protection of tidal channels.

Harbors on the South Atlantic Coast.—From New Jersey to Florida stretches the Atlantic Coast Plain. It is about 100 miles wide and is extended seaward under water 50 or 100 miles more to the edge of the continental shelf. It has a quite uniform grade to the eastward in both parts—about 10 ft. to the mile. It is almost entirely composed of fine materials, easily eroded, and furnishing, everywhere, an abundance of plastic material for the elements to mould at will.

The tidal rise varies. At Capes Hatteras and Canaveral, about 700 miles apart, it is about 2½ or 3 ft. At the middle point between them, at the bottom of the curve of the coast, it is about 7 ft. North of Hatteras it increases uniformly to 6 ft. at New York.

Wave action is moderate throughout this coast, especially on its southern portions. While the sea may be described as moderately rough at Hatteras, this decreases somewhat to the north and very much to the south, where almost the only seas of any magnitude are produced by tropical hurricanes which only last a few hours and do not produce heavy wave action.

A great ocean current, the Gulf Stream, flows north along this coast, just off the edge of the continental shelf. In the shoal waters between it and the shore just south of Hatteras, three great eddies are produced that seem to have had an influence in moulding the coast line. The principal influence, however, in shaping all this coast is the action of wind-created waves, which, while they are of a moderate character, have such an abundance of fine material to work on that their effect in changing topography is very great.

Harbors in this section have generally been improved by contraction and protection works, aided by dredging.

Harbors on the Gulf of Mexico.—The Gulf Coast has two characteristic sections. The eastern part is not much exposed to storm action, while the on-shore winds in the western part are strong and continuous. The materials in both sections are easily eroded. The tidal rise is about 1 ft.

The Gulf Stream is supposed to curve around through the Gulf, and it is possible that eddies from it have some influence on the shore-lines, but it is probably not very great.

The harbors in the eastern part have generally been improved by dredging; in the western part by contraction and protection works aided by dredging.

Harbors on the Pacific Coast.—On the southern part of the Pacific Coast, the materials are in general easily eroded and the tidal range is large, but the wind-wave action is small.

Harbors are few, the principal one, San Pedro-Wilmington, having been improved, or rather, created by dredging and works for contraction and protection.

On the North Pacific Coast movable material is abundant, though there are many rocky headlands that affect the problem. Wave action is tremendous.

There are also great ocean currents that may possibly have some effect. The tidal action is very strong and has a peculiarity that renders the maximum ebb flow much stronger than the flood.

Harbors on this part of the coast have, almost invariably, been improved by works for contraction and protection.

Methods of Improvement.

There is, perhaps, no problem presented to the engineer that is more difficult than the selection of the method of deepening the channel across an ocean bar. The prosperity of a large section of the country may depend upon its success, and, of two successful methods, one may cost many times more than the other, the sums involved being often enormous. The facts upon which the determination must be based are generally obscure. The forces to be considered are intermittent and irregular in action, and almost impossible to measure directly, either in direction or intensity. The entire topography and hydrography are frequently subject to gradual changes of great importance, but observable only in a

general way, except after long years marked by a series of expensive surveys.

These things are especially true of the sandy bars on our South Atlantic, Gulf and Pacific Coasts. The most important questions to determine in such cases are the effects of the elements upon the movable materials and the resulting effects upon the harbor entrance.

General Principles.—The following general principles are established by observation and reasoning:

The sea everywhere attacks the shore, breaking, or wearing, down larger particles to smaller ones and constantly sorting them. This work is done almost wholly by wind-created waves. These, breaking on the beach, stir up the material. Particles, fine enough to remain in suspension, make the water muddy, and may be carried by it long distances, some of it being ultimately deposited in deep water, miles off shore. Heavier particles are carried shorter distances and deposited in rougher and shallower water. Some are picked up and carried a few feet or yards and then dropped, to be again picked up. Larger ones are driven a few inches by each wave in turn.

If the waves break diagonally on the beach there will be a progress alongshore of all these particles in the direction of the component of the wind's direction parallel to the shore. The heavier particles take a zigzag path, diagonally up the beach ahead of the breaker, thence straight down the slope with the underflowing ebb.

Littoral Currents.—If the wind continues long in one direction the muddy water alongshore moves in a general current made up of the aggregate of the breaking waves. This current, of course, goes with the wind. This is properly called a littoral or alongshore current. Any such current caused by the integration of breaking waves is bound to be a powerful factor in determining the littoral drift of the sand, since it is made up of sand-disturbing units. Any other littoral current may or may not be of importance in the matter, depending on whether or not it is accompanied by some influence to stir up the sand. There are few, if any, cases in the United States waters of any tidal currents or great ocean currents, or eddies from them, or of any other style of littoral current having much influence on sand drift on ocean beaches, except when

aided by the stirring action of wind waves. This, of course, does not apply to tidal currents in or near entrances.

It should be noted that since wind waves almost always strike the coast diagonally and thereby create their own alongshore current, which, when the waves are high enough to stir up the sand, is doubtless stronger than any established general littoral current, it follows that such other littoral currents, real or imaginary, should have very little influence upon rational theories of bar improvement.

Bar-Forming Agencies.—The depth to which breaking waves may disturb the bottom is a question difficult to solve. They certainly exert force at the depth in which they break. This on our North Pacific Coast is sometimes 8 to 12 fathoms. On our Southern Coasts it is much less.

As the breaking of the waves is always shoreward it follows that there is a constant tendency to heap the sand up toward the shore. This, on the South Atlantic Coast, has resulted in a peculiar formation. The coast line is a cordon of sand islands, a few miles back of which is a similar line, and, in some sections, a third. The belts between these cordons are occupied by marsh areas or open water.

The history of these cordons is doubtless as follows: The sand driven shoreward up the gentle slope of the continental shelf gradually formed itself into an under-water ridge parallel to the shore line. As it approached the surface it was washed down by the ebb-tide. This washing was irregular as to amount and location, and ultimately the tidal escape was through low places gradually washed deeper, while the waves, unopposed, built the intervening stretches into sand islands, and the low places became entrances to the former sounds or bays. In course of time, aided probably by a rising of the whole coast, another ridge was formed from which grew the second cordon seaward of the first, and, finally, the present coast line, with its numerous entrances leading to the interior bays or sounds constituting the harbors.

The crescent-shaped bars around the entrances are submerged parts of the cordon which, were it not for the scouring action of the ebb-tide, would promptly be driven in by the waves and heaped up to close the entrances.

Across each bar is a line of deepest water or channel, which is

followed by the thread of the ebb current. This is simply an example of the tendency of the ebb outflow to concentrate in one place, just as it concentrated in the present entrances, leaving the waves to build up the intervening islands unopposed. But the concentration in the bar channel is not complete since the tide varies in volume, and a channel, suitable for an ordinary neap-tide, is insufficient for the average spring tides, and still less so for the occasional combination of a spring tide with a storm tide, hence the areas between the bar channel and the shores can only partially build up.

The Effect of Littoral Drift on Bars and Entrances.—The alongshore drift of sand above mentioned produces some remarkable phenomena when it reaches one of these entrances.

It should be stated in advance that, generally, the sand will drift in opposite directions at different times under the influence of different waves, yet there is almost always a positive resultant drift in one direction, the determination of which is the all-important element in plans for bar improvement.

When the drift reaches one of the entrances mentioned it is naturally carried along the beach into the side of the entrance, narrowing it. As the normal width and depth of the entrance must be maintained to permit the tidal flow it follows that scour will take place to restore the lost capacity. This naturally takes place from each side about equally, resulting in a net movement of the entrance in the same direction as the resultant sand drift.

The sand that is thus scoured from that side of the entrance from which the drift comes, when carried seaward by the ebb, will be deposited upon the bar, only to be brought in by the next flood or driven in by the waves. It may zigzag back and forth for a time, but, ultimately, the littoral-drift waves and the ebb-tide will work it, in a more or less continuous stream, along the bar and around the entrance.

When this sand is driven into the channel which crosses the bar it narrows it and, again, as the tide erodes about equally from each side, it follows that this channel is also driven around the bar in the same direction as the resultant drift. After this process has continued for a time the channel approaches the far shore and takes a direction approaching parallelism with it. This results in a longer route for the ebb, the slope becomes less, the channel fills up and a new channel gradually breaks out at a point nearer the

other shore, to be in turn driven round in the wake of the previous one. The period of time required for one of these cycles is measured sometimes by years, sometimes by decades and sometimes by centuries. It is a general rule that the channel is deepest and best for navigation when it occupies this break-out position, gradually deteriorating as it is driven to the lee shore. Plate XXIII shows a nearly complete cycle of this action on the Columbia River Bar.

The channel each year was north of the position it occupied the previous year and its depth gradually decreased. It will be noted that in 1902 there were three channels of about equal depth. Since that time the southernmost has begun to open out and thus start a new cycle. The elements that determine the time of a cycle are: The fineness of the sand, the force of the wind and its continuity in direction, and the shape of the bay inside, which affects the direction of ebb outflow.

The determination of the point at which the new "break-out" will occur is an important matter. The following considerations bear upon the subject. The winds that cause littoral drift are the on-shore winds. Of these there is generally a "prevailing" direction, whence comes a marked preponderance of the blows that are strong enough to stir up and transport the sands. At all events, there is, in every case, a resultant direction, generally making an angle with the coast, which is the equivalent of a "prevailing wind" from that direction. This line of direction is, at some point, at right angles to the curved crest of the bar. Should an incipient channel form on this line, the tendency to drive sand into it from either side would be a minimum. At this point, at the time when the old channel has been driven to its place of expiration, the ebb tidal flow is somewhere near to parallelism with the crest of the bar. At the same time any sands driven into the old channel at this point are quite likely to be carried further seaward by the ebb and be deposited on the bar out toward the locus of the expiring channel.

Under these conditions there is likely to open a new channel near the point where the prevailing wind is at right angles to the crest of the bar, and the direction of such channel is generally in the line of the prevailing wind.

These considerations account for the observed facts in many

cases. At Cumberland Sound Bar, Georgia and Florida, the channel swung about 1½ miles in 35 years, completing a cycle. A second cycle was completed in about 18 years, being shortened by the partial construction, meantime, of a south jetty. Both "break-outs" were on almost identical lines and directed nearly into the teeth of the prevailing wind. And in all the intermediate positions of the channel, the outer end of it, where it crossed the bar, had invariably the same direction.

These facts indicate the soundness of the following proposition, *viz., An advantageous location of the channel is at the point where the direction of the prevailing wind is at right angles to the crest line of the bar and having its axis in the same direction as the prevailing wind.*

This applies to a dredged channel as well as to one created by contraction or protection works.

Another proposition consequent upon the facts above related depends upon the tendency of the ebb flow to concentrate itself in one channel and leave the other parts of the bar with less opposition to the building-up action of the waves. It is: *If a bar channel be increased in capacity by deepening, the natural tendency is to add to that increase and to permit shoaling on other parts of the bar.*

Dredging.—The useful application of this second proposition lies principally in dredging. With the exception of Brunswick Bar, Georgia, dredging alone has never been tried on such bars in the United States to any extent large enough to produce results based on the principle stated. Small channels have been made, only to be obliterated in a short time, but nothing has been done on any scale at all proportionate to the money that has been spent on contraction works. At Brunswick, a moderate amount of dredging resulted in an increase of depth from 13 to over 18 ft., the amount removed being only about 125 000 cu. yd. Recently, the removal of 140 000 cu. yd. has resulted in an increase of about 1 ft. in depth. The first-mentioned dredging was accompanied for a time by explosions of small charges of dynamite on the bottom, but a careful analysis of the facts shows that dredging, assisted by Nature and not by dynamite, was practically responsible for the results.

In 1897, the removal by dredging of 77 000 yd. from the then

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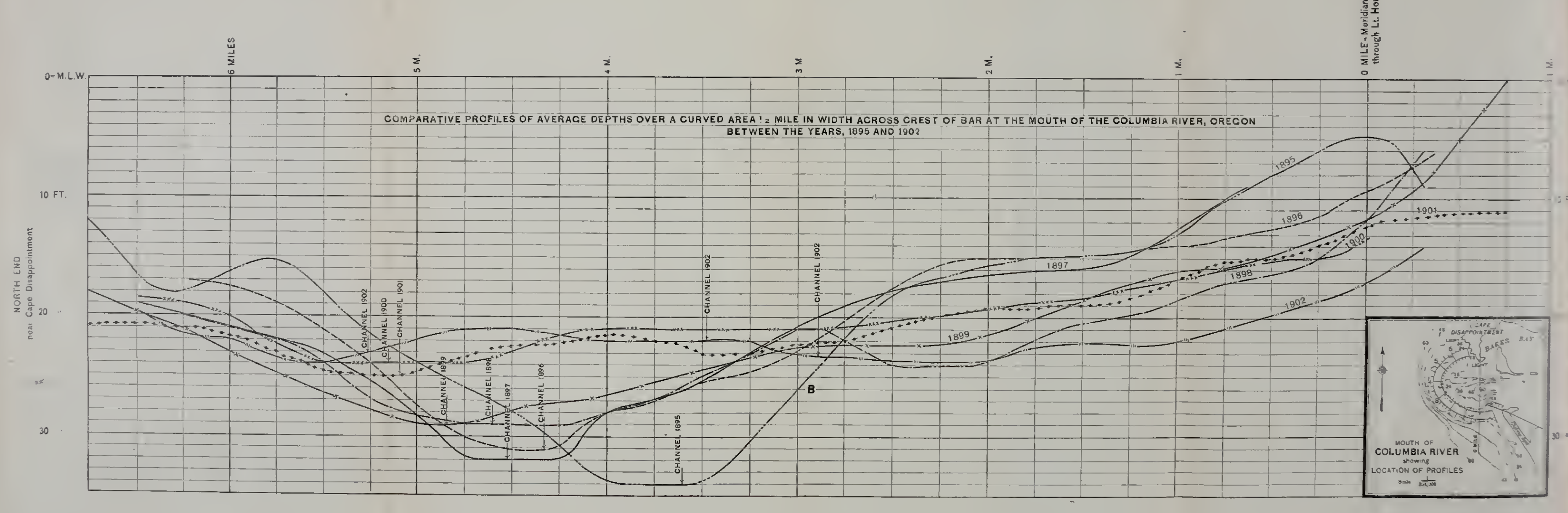


existing channel at Cumberland Sound, was followed by the natural erosion of over half that much more.

The writer believes that the future improvement of some of our South Atlantic Coast harbors can be most economically effected by dredging alone, and that, had sea-going dredges been perfected when the jetty systems of the other harbors were started, the interest on what the jetties have cost would have dredged more capacious channels than the jetties have produced.

The limits of this paper prevent the details of this argument being given for any particular case.

Contraction and Protection Works.—The existence of a navigable channel of greater or less capacity across a bar is due to the above-mentioned tendency of the ebb flow to concentrate on one line and leave the bar-building forces to decrease the depths on other parts of the bar. This channel is roughly adapted to discharge the ordinary small tide. The occasional larger tides spill over the rest of the bar, and, by opposing the bar-building waves, prevent the complete building up of those portions.

A bar channel varies as to widths and depths, according to the same laws that govern in an alluvial stream. Flowing water always tends to cut a deep, narrow channel, but this tendency is limited by the angle of repose of the material, and the channel is wider and has flatter slopes as the erosiveness of the material increases. There are eroded channels in hard rock which are deeper than they are wide, in stiff clay they are wider and shallower, and, in sand, still more so.

The side slopes of bar channels are therefore naturally very flat. There is, moreover, quite a uniformity in the materials of sand bars; hence the variations in these slopes are not wide. The depth is therefore roughly proportioned to the tidal prism. However, the exposure modifies this to a great extent. For example, Pensacola and Galveston have about the same tidal conditions, yet, in its natural state, Pensacola had a 20-ft. channel, while Galveston had only 11 ft. The difference is probably due principally to the greater exposure to wave action at Galveston, causing flatter slopes, and hence less maximum depth for a channel of the same discharge capacity.

Twin Jetties.—To increase the channel depth, the natural way

is to decrease the width of the discharge area. This is done usually by two parallel or converging jetties which concentrate the flow at the bar crossing.

Complete concentration is obtained when both jetties are built up to high water throughout and extend across the bar from the shore. Partial concentration is obtained by building to a less height, or by omitting some other portions of either or both jetties.

The difficult feature of the concentration method lies in the fact that the material scoured from between the jetties is deposited just seaward of their ends where, if the slope of the bottom is slight, it is likely to form a new bar.

Protection.—In all cases, methods of concentration involve protection. This term may mean the protection of a roadstead by a breakwater which forms an artificial harbor, usually not involving bar improvement, or it may mean the shelter of a channel.

The protection of a bar channel may take two forms. A jetty or breakwater may be used to protect dredges excavating the channel. There are no works in the United States built for this purpose only. The second form is to protect the channel against drifting sand or to prevent the degradation of its side slopes by wave action. In bar improvement by concentrating jetties, the defence of the channel against drifting sand is a vital element in the problem.

The jetty on the side from which comes the resultant sand drift is the more important. This has been called the "windward" jetty. The expression is used, theoretically, not with reference to the wind, but to the resultant sand drift. Practically it applies to the prevailing wind also, for, as indicated above, cases in which the prevailing on-shore wind and the sand drift do not agree as to direction along the coast are exceedingly rare.

The other jetty is called the "lee" jetty, but it has a similar function to perform, at such times as the sand drift is not in the resultant direction.

Location of Windward Jetty.—The correct location of this jetty is perhaps the most important thing in plans for improvement by jetties. If the amount of sand drift from that side is, as is usual, enough to fill up the navigable channel in a very few years, its principal function must be to protect the channel area from this drift.

A fact not mentioned above should here be borne in mind. There is usually at such entrances a shallow beach channel near where the shore end of the "windward" jetty must be located. This channel is formed by the flood tide as it is concentrated by the beach in its flow into the entrance. Any sand deposited in this channel is almost certain to wash into the entrance, to be carried out by the ebb and be deposited between the jetties, where it will cause endless trouble. From the fact, as herein indicated, that the drift is put in motion by the ceaseless diagonal breaking of the waves upon the beach, it would appear that the above beach channel would, under ordinary conditions, be the objective of all the sand drifting down the beach. These facts require that the windward jetty, to protect the bar channel properly, must be begun at high-water mark and be carried at that height across the beach channel and far enough seaward to impound all the drift coming from that side. This usually means that it must go to high water throughout its entire extent. In many projects and in one or two actual constructions in the United States a wide gap has been left near the shore ends of such jetties. From the above reasoning this would seem to be an error, so far as the protection of the channel is concerned, for the gap is the precise place where the sand would be most certain to be driven into the channel, where it would lessen the navigable depth or be carried seaward to form a new bar off the jetty ends.

The reasons advanced for leaving such a gap are two-fold, *viz.*, economy, and to admit the flood freely and thus secure a predominance of the ebb flow.

There can be no ultimate economy in such a plan except in a case where the sand drift is inconsiderable, and it is doubtful if such places exist. At Charleston, S. C., such a plan has been followed, and it is claimed that no appreciable amount of sand has gone through the gap. This claim is based on the fact that the bottom has not built up to the top of the sill across the gap and that no sand is ever found on the rocks of the sill after storms. Neither of these proofs is positive. Sand can easily work through a loose rock sill, where a strong current is flowing over and through the rocks, and leave no deposit on the sill itself.

The fact that the bar at Charleston has advanced manifestly

seaward and that extensive dredging is now necessary in the open sea entirely beyond the jetty ends, indicates that sand may be coming through the gap and being carried seaward. This does not indicate much economy in leaving the gap open. The facts as to the amount of sand drift at Charleston are not at hand. At Cumberland Sound it is known to be enormous, and at St. John's Bar, Florida, it is known that from 1891 to 1895 about 2 500 000 cu. yd. moved from the space just north of the partially completed jetties and was deposited between and to the south of them. It is also known that the drift on this coast is invariably to the south.

The second reason, that of ebb predominance, is worthy of careful consideration. The theory is that as little obstruction as possible should be opposed to the in-flowing flood, so as to have the maximum results from the ebb. It is claimed that a gap in the shore end of the windward jetty lets in the flood freely; but that the ebb, on account of the velocity toward the jetty channel across the bar given it by the headlands of the entrance, does not to the same extent go out through the gap and that actually more water goes out through the bar channel than comes in through it, making the predominance of the scour outward so as to avoid filling up the harbor.

While the facts on which this very neat plan is urged are doubtless true in many cases, the objection to it is, that it is not necessary. The ebb has sufficient preponderance without it. In every case the ebb is augmented by the fresh-water flow during the tidal interval, and even in cases where the fresh-water flow is insignificant the maximum ebb velocity, and hence the scouring power of the ebb, seems always to be manifestly greater than the flood.

Table 20 shows some cases of this.

The predominance of ebb-scouring power over the flood with the same tidal volume is strikingly shown by the case of Brunswick, and the entire table shows that to leave a wide gap near the shore end of a windward jetty is to invite trouble and expense for an unnecessary gain. The reason of this apparent anomaly in velocities is not far to seek. The flood pours in from an unlimited supply close at hand. Hence, though its average velocity may equal, or in some cases exceed, that of the ebb, its flow is very uniform. On the other hand the ebb flows from a limited supply from a scat-

tered area, and the discharge is irregular, so that even with no preponderance the maximum velocity must be greater than that of the flood, and it is the maximum velocity that controls the scour. On the Pacific Coast successive tides rise alternately to different heights. These in succession are, the lower low tide, the lower high tide, the higher low tide and the higher high tide; then the lower low tide. The fall from the highest stage to the lowest gives a great preponderance to ebb scour, and is very favorable to jetty improvements.

TABLE 20.

Place.	Tidal volume, in cubic feet.		Maximum velocity, in feet per second.	
	Flood.	Ebb.	Flood.	Ebb.
Savannah River (at Long Island)	2 191 000 000	2 820 000 000	2.85	3.27
Cumberland Sound (at Fort Clinch)	5 889 000 000	6 935 000 000	4.08	5.07
Brunswick, Ga	8 500 000 000	8 500 000 000	3.5	4.4
Brunswick, Ga., 1901	Same as ebb	Same as flood	5.01	6.06
St. John's Bar, Fla	1 200 000 000	1 400 000 000	4.2	6.4
Winyah Bay, S. C	....	....	3.4	7.3
Pensacola Bar, Fla	8 046 000 000	9 249 000 000	2.08	5.26
Aransas Pass, Tex	Nearly the same as the ebb.	Nearly the same as the flood.	3.9	4.3

It has been urged that closing the gap would reduce the tidal range, but the above indicates that the flood can get in through the same space rather more easily than the ebb can get out, and the tidal range should not be changed. At Cumberland Sound, where no gap was left, there was no reduction in tidal range even temporarily during rapid construction, while at Charleston, with its wide gap, the tidal range has been reduced from 5.1 to 4.9 ft. It is true that if the gap at Charleston were closed the mass of sand between the jetties would go seaward more rapidly than now, and the shallow bottom seaward of the jetties is not a favorable place to have it deposited. However, it will be carried there ultimately, with a lot more from outside the jetty. So that, in the long run, the economy of the gap is doubtful.

It should be noted that the shore end of the windward jetty is usually protected from heavy seas by a wide expanse of shoal water.

so that it need not be made of very heavy cross-section, hence the economy of leaving a gap is not very great.

It is not necessary that the entire jetty be built at once to high water, provided the high-water portion extends from the shore far enough out to impound all the drifting sand. This, however, frequently means to high water throughout.

The same reasoning applies in a less degree to the "lee" jetty which may sometimes need to be built to high water for only a short distance from the shore. In case of a practically constant drift in one direction there is not much objection to leaving a gap in the shore end of a "lee" jetty. This has been done at Aransas Pass, Texas, where a curved "lee" jetty on the north of the entrance has been under construction for a number of years and is now about completed. A gap at the shore end of this is not even provided with a sill. Apparently it has not scoured appreciably, nor is there any evidence of sand having been driven in through it, but it should be noted that the tidal range here is small, and the sand drift, while moderate in quantity, is undoubtedly to the north in the aggregate, and seems to be so in detail, seldom or never moving to the south.

The height of the windward jetty having been determined, the following considerations affect its trace. One of its vital functions being to arrest and store the drifting sand, it should, if practicable, be so located that the storage area created by it on its windward side should be a maximum.

The outer limit of this storage area will ultimately be a line parallel with the beach; but long before the sand-fill is built out to that line, sand may be going round the end of the jetty in quantity sufficient to fill up the channel or form a new bar off the ends of the jetties. A line through the end of the jetty at right angles to the "prevailing" wind will come nearer giving the proper area. When the beach has advanced to this line, sand may travel round the end of the jetty when the waves come from a point between the line of prevailing wind and the shore. The actual quantity of the resultant drift has never been estimated, some writers considering that in the general case it is hopeless to attempt to store it. The writer's opinion is that a windward jetty, whose trace makes it the equivalent of a groyne several thousand feet in length, has storage capacity for many years' drift, and that the

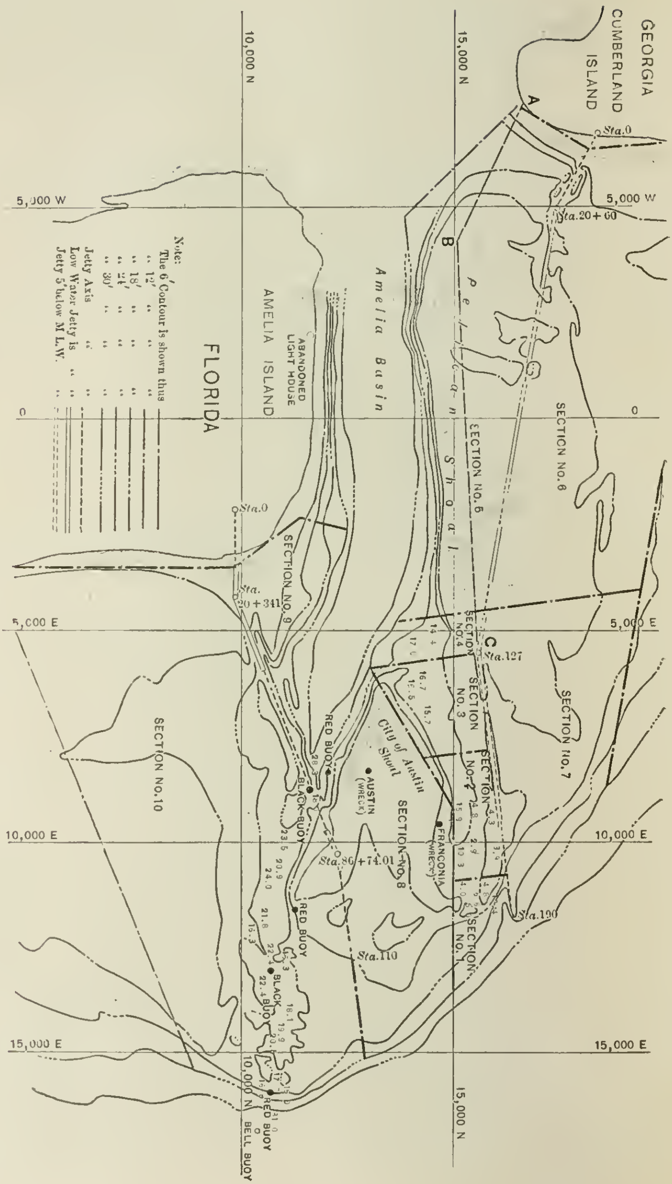
proper solution of such problems is to store the drift outside this jetty. The area rapidly increases as the jetty is extended, and the winds soon build up permanent dunes covered with vegetation, constituting a sand reservoir with proportionally unlimited storage capacity.

As usually located, a large area of sand is left between the inner half of the windward jetty and the channel. This is a bad arrangement, as some of this sand will ultimately wash seaward and make trouble. For example, in Fig. 21, the north jetty would have been in a better location had it followed the dotted line, *A B C*, around nearer the edge of Pelican Shoal. This would have given a greater sand-storing area and would have kept Pelican Shoal from wearing away so much and being deposited in the shore end of the new channel near *D* (Fig. 22).

These two figures give an example of the results of an application of the principles given above.

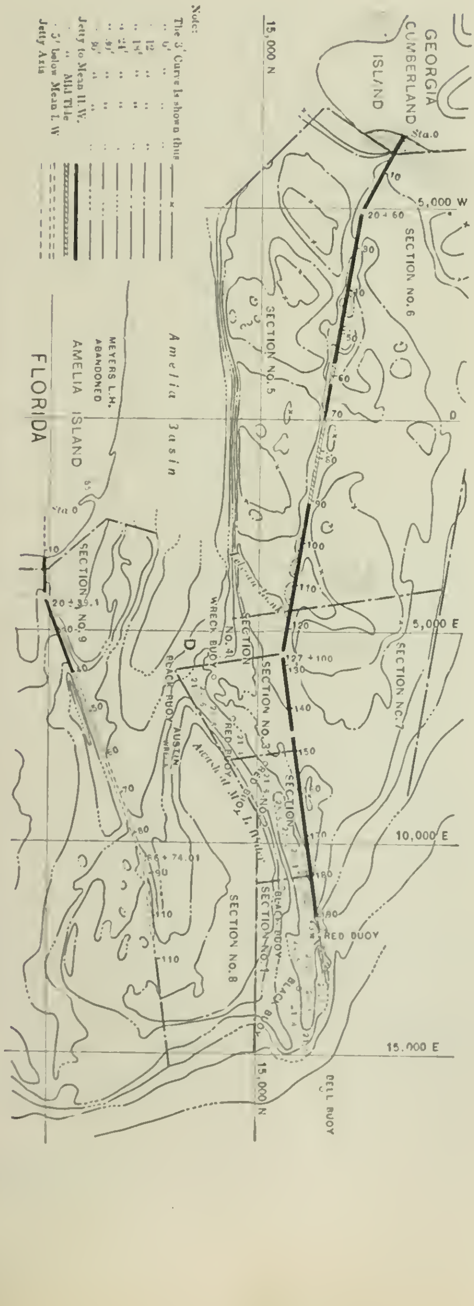
The resultant sand drift at Cumberland Sound is strongly from the north. The north jetty has been built to high water throughout. The work was begun in 1880, but progressed very slowly until December, 1900, during which time the sand drift from the north drove the channel around the bar to the south until it crossed the line of the south jetty near its middle point. This change took place without any reference to the jetties, which were then, in the parts affected, of such little volume as to have only a very slight effect on the changes of the channel. The work at that time (December, 1900), was begun by building up the outer end of the north jetty to get its protective effect upon the incipient channel near it. Work was also promptly begun on the shore end of this jetty to stop the sand from coming into the harbor from the north beach. The protection and concentration due to these works caused the channel to open promptly. In less than two years the controlling depth on the outer bar in this channel had increased from about 7 ft. at low water to over 30 ft. This depth has now maintained itself for two years. The present condition is shown in Fig. 23.

The inner end of the new channel near the point, *D* (Fig. 22), has only increased to 23 or 24 ft. at low water, due to the scouring off of that part of Pelican Shoal between the jetties. Over 1 000 000 yd. have scoured off this shoal, and a part of it has naturally lodged in the inner end of the new channel. The only remedy



ENTRANCE TO CUMBERLAND SOUND, FROM SURVEY MADE IN JUNE, 1900

Fig. 21.



ENTRANCE TO CUMBERLAND SOUND, FROM SURVEY MADE IN DECEMBER 1902

FIG. 22.

for this condition is to keep this channel open by dredging until the erosion of Pelican Shoal has ceased, or the channel opened to such an extent as to be unaffected by it. The dredging thus needed in this transition stage is probably more expensive than the better location of the north jetty, suggested above, would have been. In other respects the condition of the improvement is very satisfactory. The storage capacity north of the long north jetty is enormous. It should be many years before the sand should come round the end of the jetty in any dangerous quantity. The principal object attained by the order of work on the north jetty was to open the channel to the north of the great mass of sand which had accumulated between the lines of the jetties. The drift being to the south, this mass of sand will not travel into the channel as it would have done had the channel been opened along the south jetty. For this reason the closing of the gap in this jetty, through which the channel ran, was delayed to the last moment, and the jetty has been left short enough for the sand mass to drift freely to the south around its end.

At the same time, it must be noted that for many years the ebb-tide ran across this jetty and that the shape of the entrance, moulded to fit this condition, still gives the ebb a set this way, tending to open a second channel near the south jetty. This would be very injurious to the work, and should be combatted in its incipiency during the present transition stage. The best way to do this is to dump in the incipient channel the above sand dredged to aid the inner end of the new channel near D (Fig. 22).

These jetties do not comply with the above theory as to direction, in that they are not directed straight against the prevailing wind. They were located in 1879, and the original location has been adhered to. The north jetty has such a trace that a storm from the prevailing direction has a tendency to drive sand along the jetty shoreward rather than seaward and thus counteract the drift of the littoral current produced by the same wind. This is a favorable arrangement so far as it goes, but the shape of the bar, the quantity of sand to move, the length and cost of the jetty, etc., at this place have always been such as to prevent the adoption of the ideal trace indicated by theory.

Width Between Jetties.—The effect of twin jetties being, in such cases, made up of protection and concentration, the question of

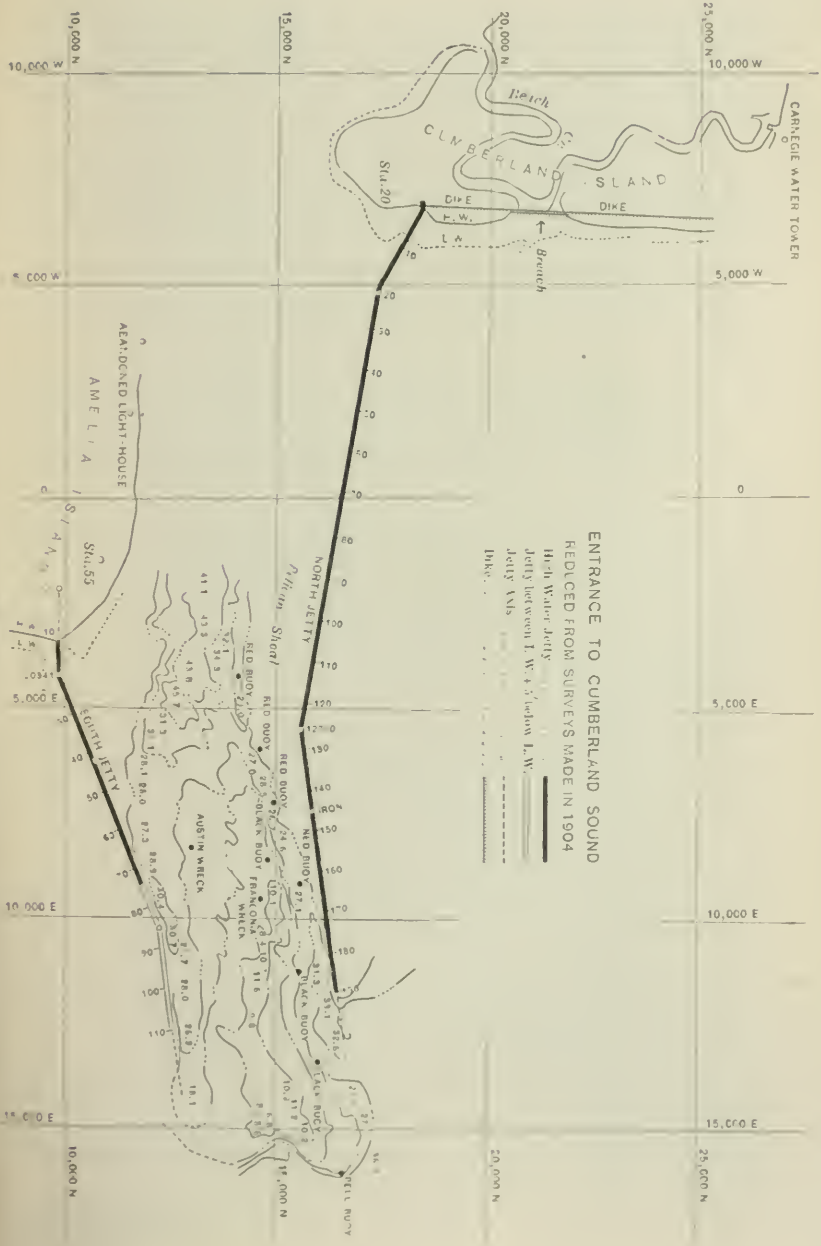

ENTRANCE TO CUMBERLAND SOUND
REDUCED FROM SURVEYS MADE IN 1904
High Water Jetty
Jetty between L.W. + 3' below L.W.
Jetty Axis
Dike
CUMBERLAND ISLAND
Beach
Breach
DIKE
H.W.
L.W.
Sta. 20
CARNEGIE WATER TOWER
AMELIA ISLAND
ABANDONED LIGHT-HOUSE
Sta. 55
NORTH JETTY
SOUTH JETTY
Pelican Shoal
RED BUOY
BLACK BUOY
AUSTIN WRECK
FRANCONIA WRECK
BELL BUOY
10,000 W
5,000 W
0
5,000 E
10,000 E
15,000 E
10,000 N
15,000 N
20,000 N
25,000 N

FIG. 23.

the proper width between them is a rather complicated one. The following elements must be taken into consideration in its solution:

a.—The tidal prism.—This requires a certain cross-section to permit the escape of the tide without creating unnecessary depths or undermining the jetties.

b.—The natural slope of the material that will exist under the conditions, as to velocity of current and exposure to waves, to be produced between the jetties after they are built.—This determines the shape the cross-section of the jetty channel will take, and from it the relations of depth to width and area of cross-section are deduced.

c.—The height and water-tightness of the jetties.—These determine what part of the water will flow out through the jetty channel.

A much used rough guide to determine the proper width between jetties is the size and shape of what is called the gorge, found at all such entrances. This is a narrow, deep channel inside the bar and usually between the ends of the islands bordering the entrance. It is caused by the erosion due to the confluence of different ebb currents and to the fact that its locus is sheltered by the bar from heavy wave action. This shelter permits the ebb river to mould its channel somewhat as it would if it were flowing through an inland sand valley.

This is indicated by Figs. 1 and 2, Plate XXIV, showing the mouth of the Columbia River before and after the jetty was built. Under the protection of this jetty, the river promptly built up its south bank, part of it to high water, at a point where there had previously been from 14 to 24 ft. at low water. Protected from wave action, this bank of the ebb river has now a steep slope where before it had very flat slopes. Part of this is of course due to the cutting off of the tidal flow over the spit, but the fact that the river cuts a channel with a steep bank, building up its own bank inside the jetty, is due to the protection of the area by the jetty.

Fig. 3, Plate XXIV, shows the condition of this entrance in 1895, showing the great improvement obtained at that time. It is to be noted from these three figures that in this improvement concentration played little or no part, as the channel flows between banks of sand having no concentrating power.

The results were accomplished by two things. The above shelter

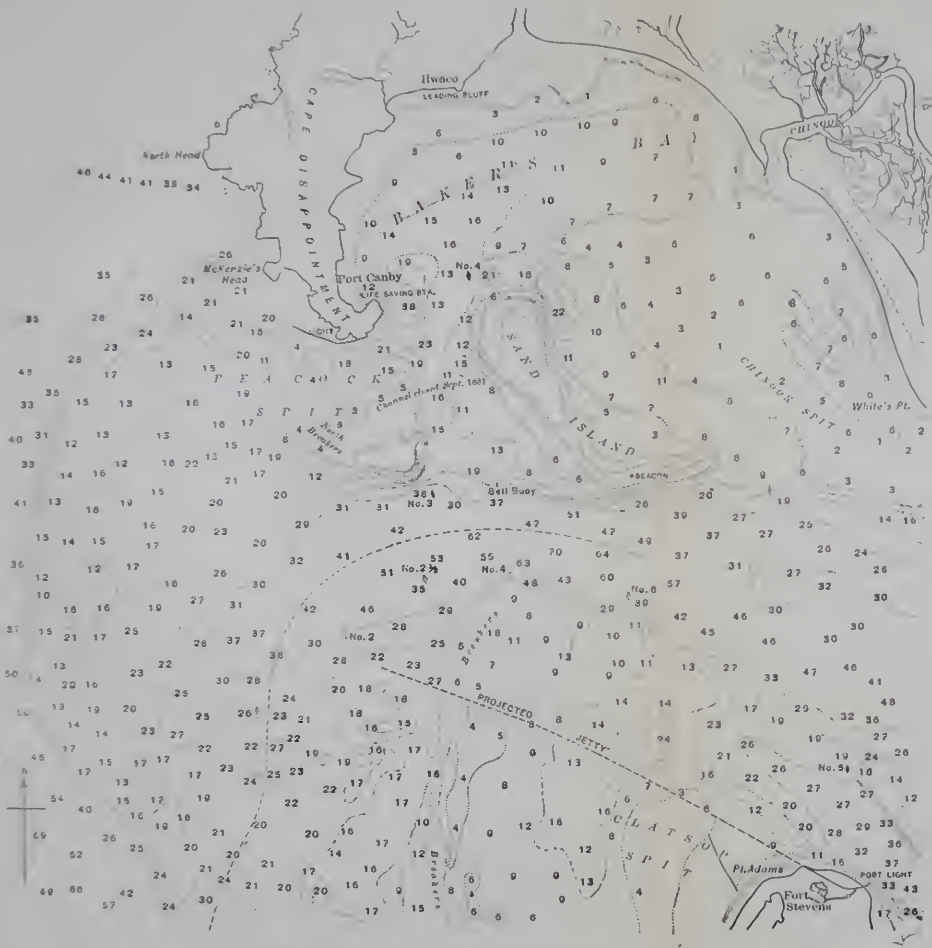


FIG. 1. MOUTH OF COLUMBIA RIVER, 1885.

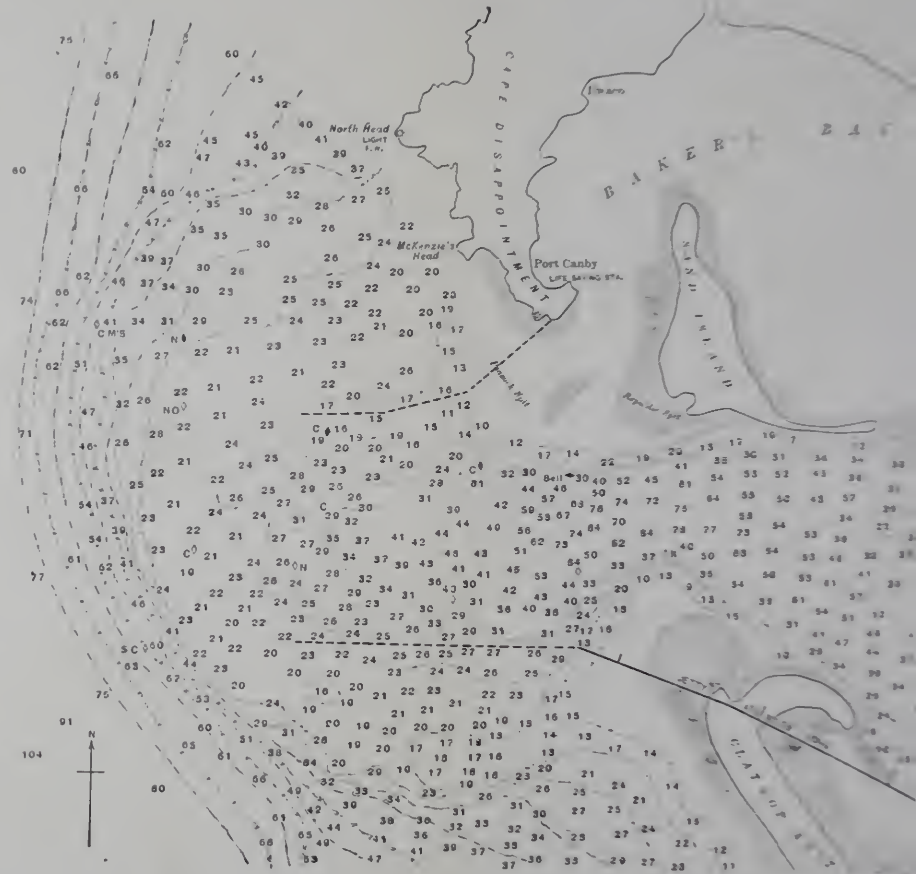


FIG. 2. MOUTH OF COLUMBIA RIVER, 1902.

FIG. 2 MOUTH OF COLUMBIA RIVER, 1902.

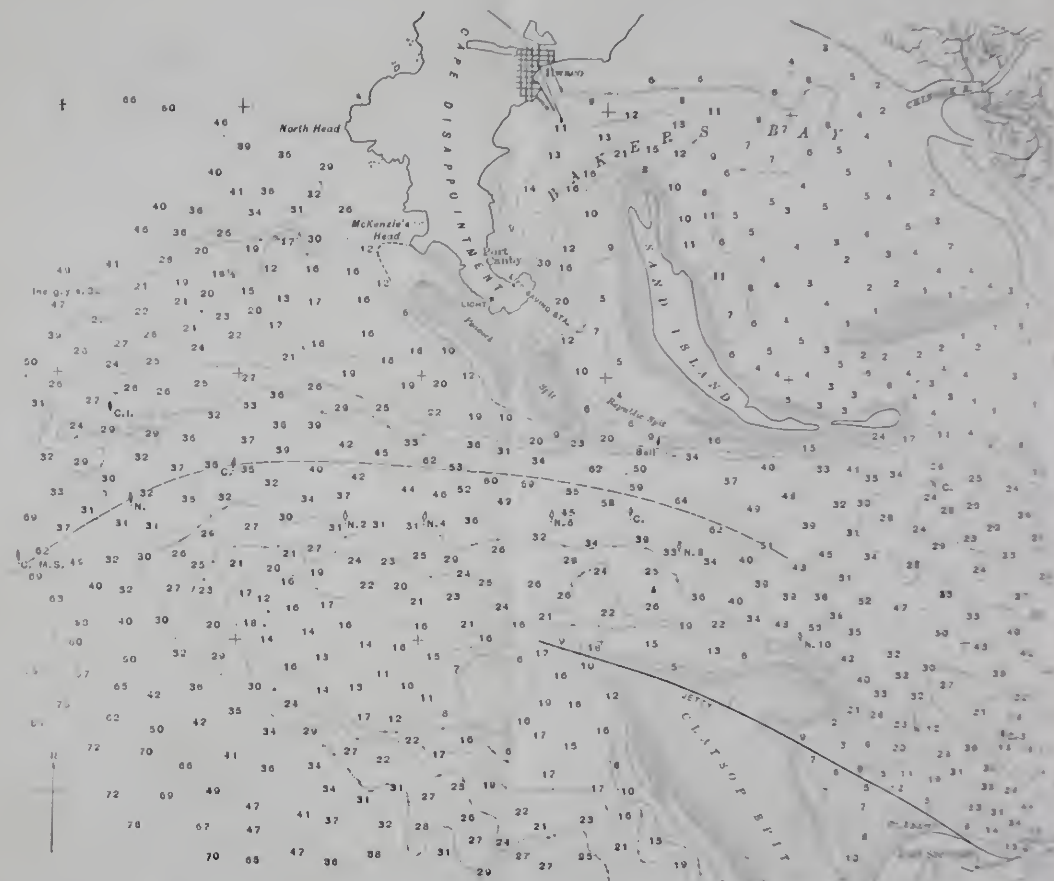


FIG. 3 MOUTH OF COLUMBIA RIVER, 1895

of the channel moved the gorge seaward and gave a more effective flow to the ebb, while the storing of the north-bound sand drift, south of the jetty, lessened the supply traveling round the bar. The travel of the sand composing the bar itself was not interrupted, so its southeast face, not receiving its usual supply, but continuing to send north its usual quota, wore thin just at the point where the assisted ebb struck it, which, breaking through, produced a fine channel. But the storage capacity south of the jetty was quite small, and as soon as it was filled (see Fig. 2, Plate XXIV), the normal sand travel was resumed and the bar was built up to its present condition.

The reason of the apparent failure at this work, following so great a success, lies in the fact that the work was stopped too soon. One-half the appropriation was returned to the Treasury. The work done is not properly a jetty at all. Originally, the shore south of the entrance was not fixed and was much drawn in from the general coast line. The work done has permanently brought it out to the general line and about even with Cape Disappointment, the opposite side of the mouth. Incidentally, this work formed a small storage basin, and produced phenomenal, but temporary, results. While the net gain in navigable depth has been little, the conditions are so improved that each linear yard of jetty to be constructed hereafter will impound more sand than many yards of the previous work. The point of the bar where additional work of improvement will be effected has not moved seaward to any appreciable extent. Under these conditions the improvement which will be effected by the extension of this jetty now going on and shown on Fig. 2, Plate XXIV, should be more extensive and very much more enduring than was effected by the little storage, incident to the bringing out of the shore line, to a point where it was ready for effective jetty improvement.

For reasons that will be given further on, the writer does not believe that what may properly be called a permanent improvement will ever be obtained by a single windward jetty acting alone.

To return to the subject of the proper width between jetties—it will be noted that the conditions as to shelter and confluence of ebb currents that exist at the gorge cannot be duplicated on the bar. Hence so great a depth cannot be expected there, and it follows

that the width between the jetties should always be greater than the width of the gorge proper. Beyond this, the gorge is not a safe guide.

The above elements of the problem of determining the proper width are so variable that the limits of this paper will not permit of the detailed discussion of even one case.

Single Jetties.—The cost of twin jetties being very great, several plans for avoiding the construction of one of them have been suggested and, in some instances, tried.

The theory on which some of them are based is as follows: The ebb flow, in its natural condition, has a spillway over the bar covering say 180° of arc. By building out a jetty in the middle of this the flow will be concentrated into about 90°, and, while the channel may swing, it should be deeper than before the jetty was built; also that somewhere on the half bar there should always be an improved channel.

The fallacy of this is that generally the channel depth and capacity depend on the ordinary small tides, and the channel, wherever located, is adjusted to this. Cutting the bar in two, therefore, limits practically the same channel to a swing through a smaller arc. The concentration is not of the main ebb flow, but of only its minor spillover, and the improvement from this cause is slight.

A Single Windward Jetty.—A jetty to windward of the channel will stop and store the drifting sand in proportion to its length, height and tightness. It does not follow that it will permanently improve the bar if it is practically complete in all these elements. Take the case of the mouth of the Columbia River (Figs. 1, 2 and 3, Plate XXIV). The river is constantly bringing down large quantities of sand, a part of which is sure to be deposited between the channel and the inner end of the jetty. This will work seaward along the jetty and, as it increases in quantity, the channel will leave the jetty and wander around the north half of the bar with about the same freedom and the same depth as it did without the jetty, except for the concentration effected by Cape Disappointment acting as a north jetty.

In the general case, of course, there is no great river to furnish the silt for this, but it must be remembered that sand can be blown by the wind over the jetty, or washed through the spaces in its structure.

Sand is also brought in with every flood-tide when the sea is rough, and, while the sand drift is assumed, in all cases, to have a resultant direction, there are, nevertheless, in nearly all cases, frequent occasions when it drifts the other way. Moreover the changes on the bar frequently cause changes and erosion inside the harbor from which a supply of sand may come to get between the channel and the jetty, and start the former on its customary wanderings. Hence, in all cases of a single windward jetty, it can confidently be expected that the channel will leave the jetty and return to its natural condition, unless there is some structure, natural or artificial, to hold it up to its work.

In the arrangement consisting of a single detached windward jetty, it can readily be seen that the drift will go through the gap and drive the channel away from the jetty long before the latter has had any chance to help develop a channel by arresting the littoral drift, for the point where the first arresting should take place is the gap itself, which, as has been shown, will not only arrest no sand, but will send it where it will inevitably prevent the operation of the theory of such a jetty.

A Single Lee Jetty.—The theory of this type of construction is that the sand drift will drive the channel to the jetty and the latter will hold it in one position where it ought to improve.

The fallacy of this is shown as follows:

The drifting sand will be constantly directed against the channel. The only thing to prevent its filling up the channel is the tidal flow which will ultimately carry the sand seaward. In time it will pass the end of the jetty, and curve round it, making the entrance impracticable and so impeding the ebb-tide that a new channel will break out farther to windward, as described in the early part of this paper. This channel will probably be a good one for a time, as is usual in such cases, and will probably not deteriorate much until it reaches the vicinity of the jetty, where, after the first cycle, it will deteriorate rapidly.

Usually such fluctuations will lessen the value of the improvement, as a fluctuating channel is little better than one equal only to its minimum depth. Another objection, especially in stormy seas, is that ships are not protected by the jetty, but are likely to be driven upon it, especially when the channel has been driven close to it. The latter condition brings with it, too, a great danger of undermining.

At Coos Bay, Oregon, this plan has been tried, with a fair measure of success. A channel depth considerably greater than called for by the project was obtained in 1894. This has, in ten years, gradually deteriorated until at present there is just the projected depth of 17 ft. at low water. Five years ago the shoal had passed to the seaward of the jetty and was working to the leeward around it, masking the direct entrance.

It is probable that the second jetty provided for in the original project will have to be built. Whether the delay in its construction has been a profitable matter is hard to tell. There has been saved the interest on its cost for ten years, and there is still a chance of saving its cost entirely. On the other hand, if a second jetty is built, there may be more sand between them than would have been the case had it been built with the other one.

At Gray's Harbor, Washington, about two-thirds of a projected single lee jetty has been built, with little improvement in channel depths. The jetty has been built to about a mile short of its projected length, and does not appear to be long enough to affect the swing of the channel. A comparison of maps indicates that since the work was begun six years ago the beach on the outside of the jetty has advanced seaward over 3 000 ft., indicating storage of a heavy sand drift from the south; and the face of the bar near the end of the jetty has scoured off, indicating an action precisely similar to that at the windward jetty at Columbia River Bar. Both these jetties are on the south sides of the channels, and they are on the same coast and only 45 miles apart, so it is not impossible that the Gray's Harbor jetty is, in reality, a windward jetty. If so, a great temporary improvement may be expected there in a year or so, especially if the jetty be lengthened to act as a more efficient sand catch. For the same reasons as urged above, the improvement would probably need a second jetty to keep the ebb channel in its proper place.

At St. John's Bar, Florida, and at Aransas Pass, Texas, twin jetties have been partially constructed, the "lee" jetty being constructed far in advance of the other as the bar was approached. In both cases the drift has passed the line of the windward jetty, which is yet low and incomplete. Very little improvement of the bar has resulted in either case.

There are some other cases of small harbors on the Gulf Coast

where partial lee jetties have been built without much success. On the other hand, high-tide twin jetties have generally been successful. At Yaquina Bay, Oregon, such jetties have created the project depth, equal to double the natural depth, and maintained it without expense for about seven years.

At Humboldt Harbor, California, twin high-tide jetties have increased the channel depth from a variable one, occasionally 25 ft., but usually from 9 to 15 ft., to a permanent one of 30 ft. and have maintained it without expense since 1899. High-tide twin jetties, with dredging, at the mouth of the Mississippi River, at Wilmington, Cal., at Sabine Pass and at Galveston, Tex., have in each case given deep water that can be depended upon. The success of the jetties at Cumberland Sound, given above, is another example.

From these facts and this reasoning it would appear that there are only two methods of bar deepening that are adapted to the conditions that obtain in United States harbors. These are dredging and twin jetties constructed either together, or with the windward one in advance, both carried to high water far enough from shore to arrest the sand drift coming from its side, the lee jetty having such length and location as to hold the ebb flow up to its work.

The choice between these two methods must depend upon the figures of cost for each particular case. If the cost of creating the channel annually by dredging is less than the interest on the cost of the permanent improvement, dredging would generally be the proper method, as the contingencies are less, and there is a probability that, once a wide and deep navigable channel is created, its maintenance would be very materially reduced, if not eliminated entirely, for years at a time.

In computing the cost of permanent improvement by twin jetties allowance must be made for the initial scour "pushing the bar seaward," a "bogey" that has seemingly alarmed many theoretical writers on the subject. There is no method yet devised, except possibly dredging, that will not "push the bar seaward," and twin jetties, being the most successful structural agency yet invented for removing the bar, are doubtless the worst sinners in this respect. The matter is not a hopeless one, however. In most cases a moderate extension of the jetties, properly aligned, will again push that part of the bar which is between the jetties into a locality where there is deep water ahead of the jetties and on both

sides of them so that the eroded sand will scatter over the bottom and disappear.

It should be noted, moreover, that the value of an improvement is its reliability, and that the increased value of the products of the country tributary to the great majority of our seacoast harbors is worth many times the cost of complete and sure works of improvement.

Determination of the Direction of the Resultant Sand Drift.—It has been shown that the resultant drift is a most important matter. It is, however, in many cases, a most difficult matter to determine, since every known indication of it sometimes either fails, or is not available. These indications are as follows:

1.—The Swing of the Bar Channel.—This has been explained in detail. It is a sure sign when its records are available. This requires many maps, and they are usually available only at places where a plan of improvement has already been adopted and the works put under construction. Oral testimony is generally unreliable, as, with changing shores and ancient memories of such difficult things as one's location on the water, such testimony is bound to be uncertain or erroneous.

In interpreting maps, too, there must be maps enough to determine whether the channel swung, or jumped to its new location.

2.—The Movement of the Entrance.—This in general is a good indication, but it sometimes fails. Some entrances cannot move on account of natural hard materials, others move by jumps against the sand drift, and occasionally one will drift against it owing to peculiar conditions. Thus, at Aransas Pass, Texas, the entrance has moved south a long distance against the resultant drift, and work is now going on there, on the theory that the drift is from the north, whereas it appears to be distinctly from the south.

3.—Configuration of the Beach.—The waves breaking diagonally on shore wear it away and move it forward. The beach is not all of a uniform texture. The harder spots will remain as points, and the spaces between will wear faster. A moment's reflection will show that the resulting shore line will consist of a series of curves, with a short curve on the lee side of each obstacle or hard spot and a long curve on its windward side. Any long curve prolonged will pass seaward of the next long curve on its lee. This is a good indication, but is likely to change for different seasons. If it indicates the same direction for a year, it is probably a safe guide.

4.—Stream Deflection.—The mouths of small inlets all turning one way on the same coast are useful as a check, but are subject to the same irregularities as the last.

5.—Wind Records.—The sand drift being almost invariably with the wind, the record of such winds as cause breakers strong enough to stir up the sand is, perhaps, the best-known index of the sand movement. Such records are kept by the Weather Bureau.

From a careful study of the above facts, in any particular case, a safe deduction as to the direction of the resultant drift can generally be made.

Mouth of the Mississippi River.—This forms one of our most important harbors, and has a regimen peculiar to itself, not covered in some particulars by the above analysis.

It carries an enormous quantity of silt. It is not affected materially by the small tide that exists in the Gulf. Its banks are of soft mud. It has several mouths. Twin jetties, aided very materially by dredging, have constituted the plan of its improvement. The conditions above given have prevented much concentration. If this were attempted, the necessary contraction would simply lessen the flow from the mouth under improvement and increase it from the others. This has been partly remedied in the past by sills at the heads of the unimproved passes to limit their enlargement. The littoral drift here is probably to the west, although the mouth of the river apparently throughout the entire delta has moved almost constantly to the east. This is probably due to the fact that the strong current out of the river into the almost tideless Gulf projects the mud-laden water a long distance from shore, and the general drift of the water takes it thence to the westward, where it settles and shoals the bottom off the western mouths, leaving it deeper off the eastern ones. Under these circumstances the western mouths close and an easterly one becomes the main river. This hypothesis is somewhat confirmed by the fact that in its lower fifty miles, projecting into the Gulf, the strip of land forming the western bank is everywhere twice as wide as the corresponding strip on the east. This would indicate the possibility that longer usefulness might be looked for if for the present improvement a more easterly mouth, which was otherwise as good as the South West Pass, could have been found.

Alignment of Jetties.—As jetties are much cheaper if they can be kept some distance from the channel, it would appear that refinements in alignment are not necessary. Straight or broken lines are much simpler to follow in construction, and the best practice now is to avoid curves except to connect lines not making very obtuse angles. Curves to affect scour, are useless. The boring action of a concave bend in a river cannot be imitated on an ocean bar because such effect is only produced too close to the bank for the safety of the structure and much too close for the use of shipping.

Construction of Jetties.—Practically all the jetties in the United States have been built of rubble stone. In the past, brushwork has been used as a foundation, to prevent scour and settlement. Since about 1896, this has been generally abandoned, it being found that a layer of small stones 2 or 3 ft. thick answers every requirement and is very much cheaper. A number of years ago log and brush foundations were used. They were found to be very unsatisfactory, from irregular settlement and destruction by the teredo, and were long ago abandoned.

On the South Atlantic and Gulf Coasts, the water is smooth enough to make it practicable to build jetties from barges. On the North Pacific Coast, it is necessary to use double-track railroads built on trestles along the site of the jetty, driving the piling in advance from the trestle itself and dumping the rock from cars.

Protection of Jetties Against Undermining.—Should a channel approach too near a jetty, it is necessary to protect the latter. The best practice is to rip-rap the bottom alongside the jetty with a layer of small stone, 50 or 60 ft. wide. This has worked perfectly at Cumberland Sound. On the Pacific Coast, this method is probably a little awkward to apply from a trestle, and groynes built from spur tracks have been used. They have usually answered the purpose, but, at least in theory, they are not nearly so good as the rip-rap method. A groyne sets up an eddy on one side when the tide ebbs and on the other in flood. This may cause dangerous scour unless long and expensive groynes are used. This is impracticable if the jetties are located the proper distance apart, but groynes are useful if the opening requires reduction.

TRANSACTIONS

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Paper No. 15.

HARBORS.

THE DELAWARE, SANDY BAY AND SAN PEDRO BREAKWATERS.*

By C. H. McKinstry, M. Am. Soc. C. E.†

It is propcsed to give brief descriptions of these breakwaters and to add a few remarks on the type of construction which they exemplify.

The Delaware Breakwaters.

The relation of these breakwaters to the shore lines in their neighborhood will appear from an inspection of Plate XXV.

The Old Delaware Breakwater.—The old Delaware breakwater consists of three parts. The easterly arm, 2 558 ft. long, and the westerly arm, 1 358 ft. long, formerly called the "breakwater" and the "ice breaker," respectively, were built between 1828 and 1869. The gap between them, 1 350 ft. long, was closed between 1884 and 1898. The "breakwater" and the "ice breaker" are rubble mounds rising to the height of from 12 to 14 ft. above mean low water (see Figs. 1 and 3, Plate XXV). The quantity of stone deposited in the two works was 892 528 long tons, at a cost of $2 192 103.70, in-

* Data as to Delaware and Sandy Bay breakwaters furnished by Colonel Stanton and Major Sanford, Corps of Engineers, at present in charge of these works, respectively.

† Captain, Corps of Engineers, U. S. A.

cluding expenses of all kinds, or $2.45 per ton. Stones weighing from ¼ ton to 7 tons were used, the larger stones being placed on the top and slopes.

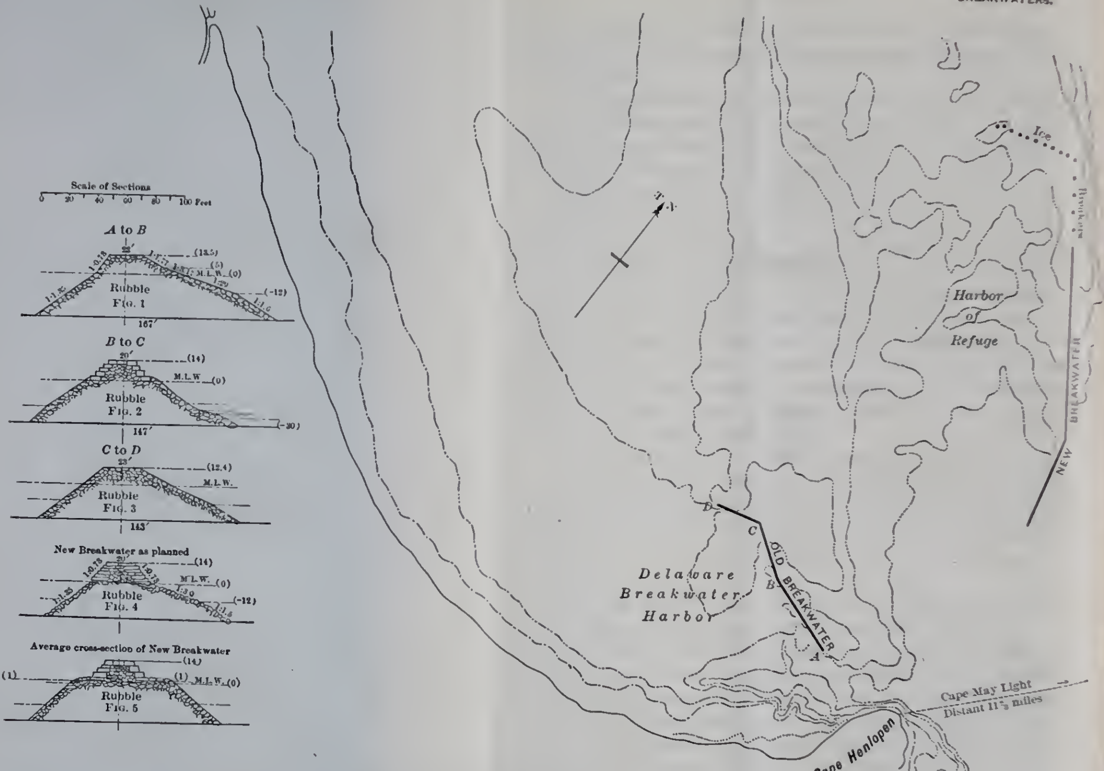
Long before these works had been completed the need of closing the gap between them had become evident. Many interesting projects for the connecting work were proposed, in several of which the influence of foreign constructions is clearly to be traced.

The first plan was for a rubble substructure rising to 12 ft. below mean low water, surmounted by a superstructure of concrete blocks built in place, each block to be 16 ft. long, 24 ft. high, 24 ft. wide at the bottom and 12 ft. wide at the top. Before work was begun this plan was modified to provide for a brush mattress foundation. By 1889 the mattress foundation and about 100 000 tons of substructure stone had been placed. In the following year it was decided to stop the substructure at 15 ft. below mean low water and to build thereon a concrete superstructure on the sloping-block system. In 1891 the plan was changed to the one which was finally carried out (see Fig. 2, Plate XXV).

As this final plan has exercised a great influence on the design of other American breakwaters, the considerations which led to its adoption will be briefly set forth. They are explained in full in an interesting report* by Colonel (now General) C. W. Raymond, M. Am. Soc. C. E., under whose charge the work in the gap was completed.

With respect to the upper portion of the "breakwater" it was noticed that in many places slopes steeper than 1 on 1 had been stable for years. It was inferred that for the superstructure of the connecting work flatter slopes than these would not be necessary. They could be most conveniently, and, so far as quantity of stone was concerned, most economically, formed by building the superstructure of two walls of roughly rectangular stones, laid as headers, with the space between compactly filled with rubble. As to the portion of the old work below low water, it appeared that possibly an unnecessarily large quantity of material had been used. The factors that had controlled the substructure slopes were the action of the sea, the size of the blocks used, and the place of deposit of the stone. It was apparent that a substructure of minimum volume and

* Annual Report of Chief of Engineers, U. S. A., for 1899, p. 1 346 *et seq.*



DELAWARE BREAKWATERS

Scale of Feet

1000 500 0 1000 2000 3000 4000 5000

steepest stable slopes could be formed by depositing all material within a given width, this being the width required to accommodate the superstructure, and by supplying material until the effect of the waves upon the slopes had ceased. In passing, it should be remarked that this mode of construction is suitable for places where there is a yearly stormy period, and not for places where severe storms occur only at intervals of several years, as at San Pedro, Cal.

The proper height of the superstructure was assumed as 14 ft. above mean low water, and the proper top width as 20 ft. Side slopes of 1 on 0.7 were assumed, giving a width at mean low water of about 40 ft. The substructure was to be built by the deposition of stone within this low-water width, the larger stones being dumped on the sea side. For purposes of estimate, a slope of 1 on 1.35 was assumed for the harbor side of the substructure, and for the sea side a slope of 1 on 3 to 12 ft. below mean low water, and 1 on 1.5 thence to the bottom.

Fig. 2, Plate XXV, shows the average cross-section of the work as constructed.

The superstructure has been stable, but it has been found necessary to protect the top of the substructure by heavy stones laid along the toe of the sea wall of the superstructure. The harbor slope of the substructure is slightly flatter than was figured on. The ocean slope averages 1 on 1.5.

The total quantity of stone used in closing the gap was 207 103 tons, and the work cost $615 036, or $2.97 per ton.

From the behavior of this work it is inferable that for localities having an equal exposure a superstructure of the form and construction there used, but considerably less wide, is amply strong; that if the harbor side of the substructure is formed by the deposit of large stones, a slope of 1 on 1.3 or even steeper will be stable; and that on the sea side a slope of 1 on 1.5 will be stable if formed by the deposit of stones weighing from 4 to 15 tons. Evidently, below a certain depth, say, 12 ft. for such exposure, a steeper slope will be permissible. Since, when the deposit of material for the substructure is confined within the width of the base of the superstructure, the wave-imposed slope may take a long time to form, it is evidently advisable to form the 1-on-1.5 slope by the deposit of stone outside the limits mentioned.

The earlier work on the old Delaware breakwater was accompanied by marked shoaling of the sheltered area. By 1882 this shoaling had averaged between 4 and 5 ft. With the closing of the gap, scour was observed, especially just inside the eastern end of the "breakwater."

The New Delaware Breakwater.—Due to the continued increase in the size of vessels needing shelter at this point, a strong public demand was made for the construction nearby of a harbor of refuge of the first class. In 1892 a plan therefor was submitted by a board of Engineer Officers convened by authority of Congress. The River and Harbor Act of June 3d, 1896, adopted the plan of this board and authorized the Secretary of War to enter into contract for the completion of the work at a cost not to exceed $4 665 000. The breakwater and ice piers constructed in accordance with this Act are shown on Plate XXV.

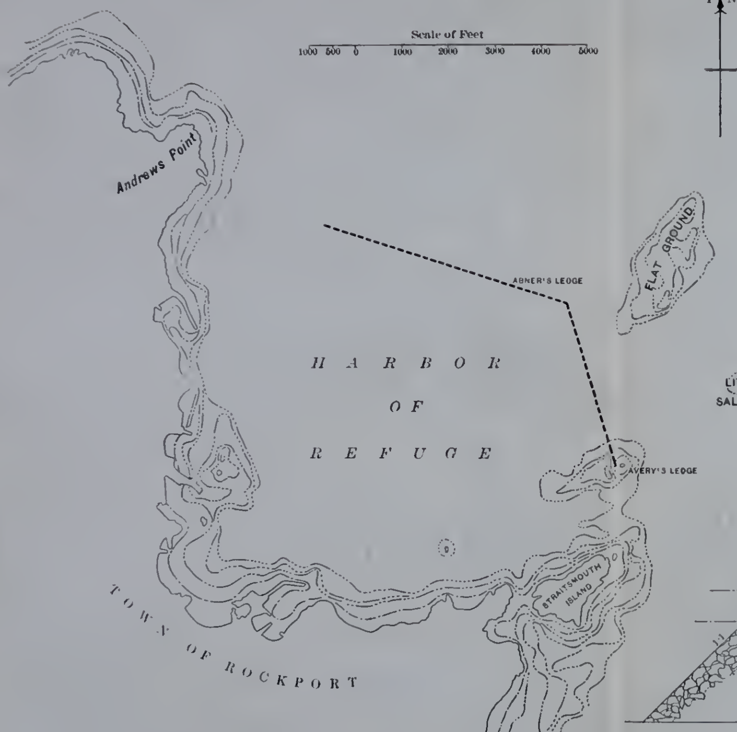
The breakwater is 8 040 ft. long, measured at the low-water line, the northern arm being 5 360 ft. long and the southern, 2 680 ft., and it contains 1 475 276 tons of stone, 1 210 133 in the substructure, and 265 143 in the superstructure. Work on the breakwater began May 3d, 1897, and was completed December 11th, 1901. The contract price for both sub- and superstructure was $1.18⅗ per ton. The northerly ten ice piers (the ones proposed by the board) were built in 1901-02. They contain 71 300 tons of stone, the contract price for which was $2.25 per ton. Five additional ice piers were built in 1902-03. The contract price was $2.89 per ton, and they contain 57 673 tons. In construction, the ice piers are similar to the breakwater. Their top dimensions are 15 by 18 ft. The stone for the superstructure of the five piers last built was brought from Maine.

The stone used in the breakwater is a very dark granite, hard and durable, weighing from 167 to 170 lb. per cu. ft., and was quarried at Bellevue, Del., about ¾ mile back from the Delaware River, and 78 miles from the site of the breakwater.

From the quarry it was brought on flat cars to the river, where it was loaded on deck and dump barges and towed to its destination. The capacity of these barges ranged from 550 to 1 400 tons, and 1 273 barge loads were delivered.

In building the substructure the stone was brought to a plane of 12 ft. below mean low water by the use of bottom-dumping scows. Above this level, all stone was deposited by derricks.

SANDY BAY, MASS.



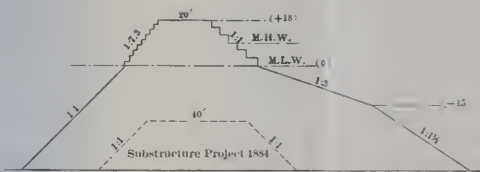


FIG. 1 Section adopted 1892

Scale of Sections
10 5 0 10 20 30 40 50 feet

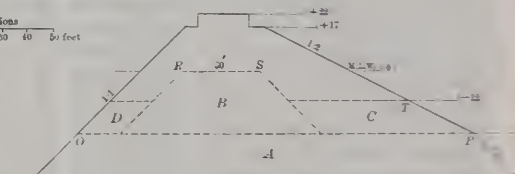


FIG. 3

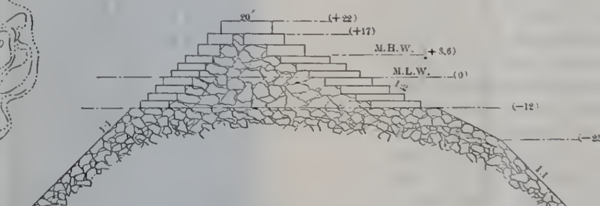


FIG. 2 Section adopted 1902

The superstructure was built by means of derrick barges with long booms, the barges lying on the harbor side of the breakwater. A traveling crane was used for part of the work, but proved unsatisfactory. The walls were laid dry in courses, the stones being roughly dressed to give a fair bearing surface. Superstructure was not built on substructure that had been in place less than one winter season.

The stones in the substructure and core of the superstructure range in weight from 25 lb. to 25 tons. The weights of the stones forming the walls of the superstructure range from 5 to as much as 23 tons, the largest stones being in the outside wall.

The average width of the substructure at the level of mean low water is about 77 ft.; the superstructure is 41 ft. wide at the bottom, 20 ft. wide at the top, and 14 ft. high. The substructure is carried up above the bottom of the superstructure about 4½ ft. on the outside and 2½ ft. on the inside, forming berms about 25 and 14 ft. wide on the outside and inside, respectively. The average slope of the substructure is about 1 on 1.2, and of the superstructure about 1 on 0.9 (see Fig. 5, Plate XXV).

The low-water depths in which the breakwater was built varied from 9.6 to 55.1 ft.

The greatest quantity of stone deposited in one month was 62 700 tons in June, 1899. The greatest amount of work done on the superstructure in one month (July, 1901) was equivalent to 531 ft. of completed work.

The cost of the breakwater and the fifteen ice piers was about $2 240 000, or less than one-half the estimated and authorized cost.

The breakwater protects an anchorage of about 798 acres from southeast to northeast, the sector of heaviest winds and seas, and the ice piers protect it sufficiently from ice brought down by the bay on the ebb tide; but experience shows that the winds and seas from the northwest are at times violent enough to make the anchorage dangerous. Under authority of the River and Harbor Act of June 13th, 1902, a plan has been recommended by the Chief of Engineers, U. S. A., for completing the protection of the anchorage area. It involves closing the spaces between the ice piers, and building, in a southwesterly direction from the northwesterly ice pier, an arm about 10 000 ft. long, or even to the shore if that should be found desirable as the work progresses. From the northerly end of the present work

to the fifth ice pier, the section proposed is the same as that of the present work. The section proposed for the remainder is to be of the same form, but 9 ft. high above low water and 10 ft. wide at the top.

The Sandy Bay Breakwater, Massachusetts.

The subject of an extensive harbor of refuge at Sandy Bay has been under consideration since 1882, in which year Congress provided for an examination and survey of the bay "with a view to the construction of a breakwater for a harbor of refuge." The project submitted in accordance with this Act was for the construction, at a cost of \$4 000 000, of a breakwater, 9 000 ft. long, in the location shown on Plate XXVI. The proposed breakwater was to be a rubble mound surmounted by a masonry superstructure founded 15 ft. below low water. The mound was to be 40 ft. wide at the top. The superstructure was to be trapezoidal in section, to rise 8 ft. above high water, and to be 15 ft. wide at top. Below low water, it was to be laid "dry"; above low water, in mortar.

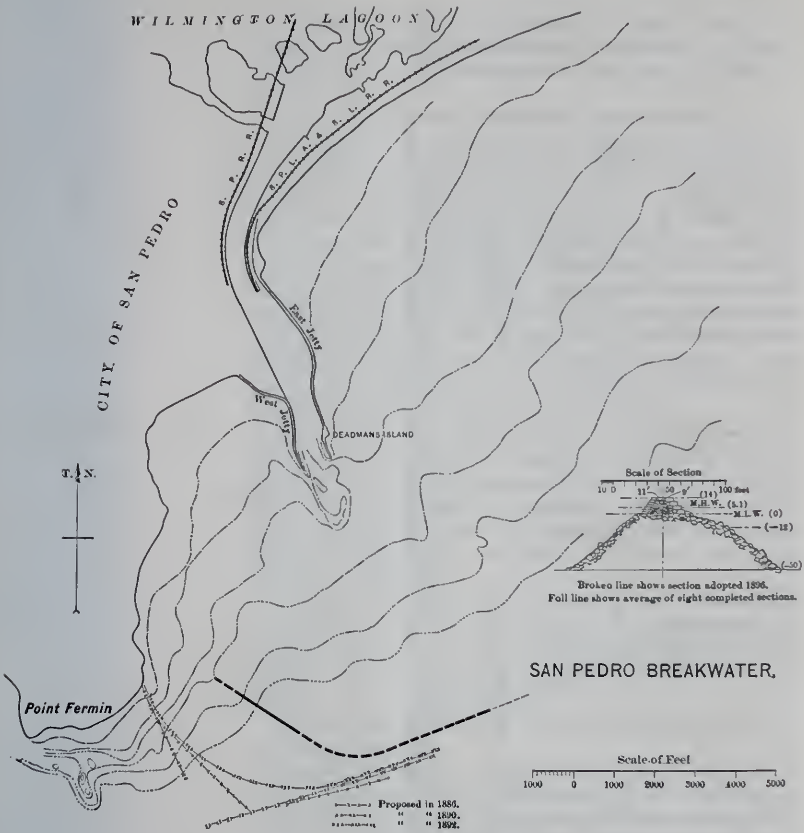
In 1884 the above-mentioned estimate of cost was increased to \$5 000 000 by a board appointed pursuant to the River and Harbor Act of that year. This Act contained the first appropriation for the work, \$100 000, but made its expenditure contingent upon the board's reporting that Sandy Bay was the best place for a harbor of refuge between Boston and Portland. The board's report was favorable to Sandy Bay.

No work was ever done upon the superstructure above described, and, in fact, no project for the construction of a superstructure was adopted until 1892. In 1884 the plan for the substructure was changed to that of a mound, 40 ft. wide at top, rising to 22 ft., instead of to 15 ft., below low water.

The location originally proposed for the breakwater has, however, been adhered to.

The depth of water at mean low water varies from 6 ft. at Averys Ledge, the extreme southerly end of the breakwater, to about 89 ft. at the extreme westerly end, and averages about 45 ft. along the southerly arm, and about 65 ft. along the westerly arm. The bottom along the line of the work is nearly all ledge, except at the westerly end, where it is sand and shells. In the anchorage area, the holding-ground is excellent, being sand mixed with mud.

PLATE XXVII. VOL. LIV. PART A.
TRANS. AM. SOC. CIV. ENGRS.
INTER. ENG. CONG., 1904.
McKINSTRY ON
BREAKWATERS.



The work done prior to 1892, up to which time $450 000 had been appropriated, consisted in the placing of about 500 000 tons of stone in the substructure.

In the early part of 1892, a board was appointed to recommend a project for the superstructure and any changes that might be desirable in the existing project for the substructure. The section recommended by this board and adopted March 2d, 1892, is shown in Fig. 1, Plate XXVI. This section is practically the one used in estimating the cost of closing the gap in the old Delaware breakwater (see Fig. 2, Plate XXV).

By 1898, 600 ft. at the northerly end of the southerly arm had been completed to full section, 1 200 ft. more had been carried up to low water, and 2 800 ft. more had been built as shown by the broken line in Fig. 1, Plate XXVI. The 600 ft. of superstructure were formed of stones weighing not less than 4 tons each and averaging 6 tons, and the southerly 250 ft. of it had settled some 2 ft. In the early part of the year, in a storm of exceptional severity, the 600 ft. of completed superstructure was torn down to a height of about 5 ft. above mean low water.

The appropriation of 1899 ($250 000) was coupled with the proviso that a board should be appointed to report whether any modification of the project should be made, with an estimate of the cost of completing the breakwater.

The modifications recommended by the board and adopted September 18th, 1902, will appear from a comparison of Figs. 1 and 2, Plate XXVI.

The capstones in the new plan are to weigh not less than 20 tons, to be 20 ft. long by 3 by 5 ft. in end-section, laid on edge and in as close contact as possible. The course below the capstones is to contain two stones, each weighing about 10 tons, the outer stone to be at least 15 ft. long and the inner one at least 10 ft. Below this course to a depth of 12 ft. below mean low water, the stones in the outer face weigh at least 8 tons, and in the inner face at least 3 tons; and all are to be laid horizontal and as headers. The board estimated that the cost of completing the work to the section recommended would be $5 891 832.70. Including what had been spent to that time, the total estimate for the work was $6 904 952.25.

The River and Harbor Act of June 13th, 1902, appropriated

\$200 000 for continuing the improvement, but provided that no part of this sum should be expended until a board had examined the project and reported upon the feasibility and advisability of continuing it to completion and the possibility of so modifying it as to reduce the cost. This board reported, September 18th, 1902, that the present project (Fig. 2, Plate XXVI) was entirely feasible, that the completion of the work was desirable, and that no modification that would reduce its cost was possible.

About one-half of the stone in this breakwater has been deposited from dump scows and the remainder by derricks.

In working under the present project, Section *A* (see Fig. 3, Plate XXVI) is first formed, the dumping being restricted to within the crest line, *O P*, of that section, the stone being allowed to take its natural slope on either side. Section *B* is next formed, the dumping being again restricted to the crest width, *R S*, and the stone being allowed to take its natural slope. Section *C* is next deposited, the exterior slope of 1 on 2 being formed by placing large stones with the derrick. No work will be done in Section *D* until the placing of the facing-blocks is about ready to begin. No work as yet has been done except in Sections *A*, *B* and *C*.

The present condition of the work (September, 1904) is as follows: 2 900 lin. ft. of the southern arm, measured from a point 70 ft. north of the end of the work on Averys Ledge, has been built to the full area of the cross-sections, *A*, *B* and *C*. Thence to the angle, the same section obtains except that Section *B* has been carried to 5 ft. above mean low water. West of the angle, 400 ft. has been completed to the combined cross-section, *A*, *B* and *C*, and Section *B* has been carried to the further height of mean high water (+ 8.6 ft.); thence, 1 050 ft. has been built to the combined cross-section, *A*, *B* and *C*; thence 50 ft. of Section *A* has been built and, for 250 ft. further, some stone has been deposited.

It is estimated that when completed the work will contain 6 301 407 short tons of stone. To September, 1904, 1 690 178 have been deposited.

In respect to appropriations, this work is the antithesis of the new Delaware and San Pedro breakwaters. The first Acts of appropriation for the last-named works authorized the Secretary of War to enter into contract for the completion of the projects. Ap-

FIG. 1.—TRESTLE AND CRANE, SAN PEDRO BREAKWATER, 1904.

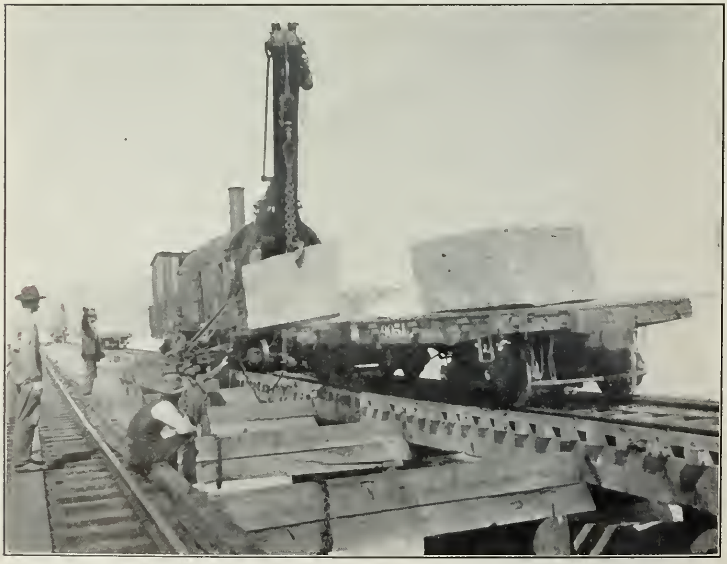


FIG. 2.—TWO STONES FOR OCEAN WALL OF SUPERSTRUCTURE OF SAN PEDRO BREAKWATER, 1904.

propriations for Sandy Bay began in 1884 and aggregate $1 372 000, an average of $68 000 per year, or about 1% of the total estimated cost.

The San Pedro Breakwater, California.

The River and Harbor Act of 1886 provided for an examination and survey of "San Pedro Bay near the entrance to Wilmington Harbor with a view to establishing an outer harbor for the protection of deep-draft vessels." The report on the survey contains a project for two breakwaters (see Plate XXVII), to cost about $4 000 000.

The Act of 1890 directed the appointment of a board of three officers of the Corps of Engineers, U. S. A., to examine the coast between Points Dume and Capistrano, and to select the best location for a deep-water harbor, submitting, with their report, a project and estimate of cost. This board reported that the best location for such a harbor was found at the western end of San Pedro Bay. They proposed the construction of two breakwaters (see Plate XXVII), at an estimated cost of about $4 500 000.

The River and Harbor Act of July 13th, 1892, directed the appointment of a board of five officers of the Corps of Engineers, U. S. A., to make a careful examination of San Pedro and Santa Monica Bays, to report as to which was the more eligible location for a harbor to accommodate the "largest ocean-going vessels and the commercial and naval necessities of the country," and to submit, with their report, a project and estimate of cost. This board, like the preceding one, reported in favor of San Pedro, and recommended the construction of a single breakwater attached to Point Fermin (see Plate XXVII), at an estimated cost of $2 885 000. The reasons given for the finding in favor of San Pedro were that the natural protection afforded by the configuration of the shore was greater at San Pedro Bay, and that there was already at San Pedro an interior harbor on which the United States had spent considerable money, and which would be a valuable adjunct to the outer harbor created by the breakwater.

The River and Harbor Act of June 3d, 1896, provided for the appointment of a board to consist of an officer of the Navy, an officer of the Coast and Geodetic Survey, and three civilian engineers, who should examine Santa Monica and San Pedro Bays and report which of the two offered the more desirable location for

a deep-water harbor. It was further provided that the decision of this board should be final as to the location of the harbor, and that the Secretary of War, upon the receipt of the finding of the board, with plans, specifications, and estimates, should enter into contract for the construction of the breakwater recommended, at a cost not to exceed \$2 900 000. This board repeated the finding of the previous boards as to location, basing its verdict in favor of San Pedro upon substantially the same reasons. The location and type of the breakwater adopted are shown on Plate XXVII. The breakwater is to be 8 500 ft. long, or as much longer as may be constructed with \$2 900 000.

On August 12th, 1898, a contract was entered into for the construction of the breakwater. The contract prices were: For substructure, \$0.54½ per long ton; for superstructure, \$0.72 per long ton; for the concrete in the monoliths which are to form the ends of the superstructure, \$6.80 per cu. yd. Using the board's estimated quantities, the cost would be \$1 303 198.54. This contract was annulled on March 19th, 1900, the contractors having failed to attain the rate of progress required by the specifications. Under this contract 84 581 tons of stone were deposited in the substructure. On June 7th, 1900, a contract was entered into with the California Construction Company of San Francisco, Cal., for completing the structure. The contract prices are: For substructure stone, \$0.844 per long ton; for superstructure stone, \$3.10 per long ton; for concrete, \$6.00 per cu. yd. Total consideration, using the board's quantities, \$2 375 546.05.

To September 1st, 1904, 1 685 710 tons of stone had been deposited under this contract, 1 638 799 tons in the substructure and 46 911 tons in the superstructure. The substructure is about completed for a length of 7 600 ft., and the work done on the superstructure is equivalent to 1 584 lin. ft. of completed work. The greatest quantity of stone placed in any one month was 47 258 tons in May, 1904.

Under the first contract, stone was brought from Catalina Island, 22 miles distant, and was deposited from self-dumping barges. Under the present contract, the trestle method is used. Two kinds of stone are used for the substructure, *viz.*, sandstone from Chatsworth Park, 60 miles from the site of the breakwater, and

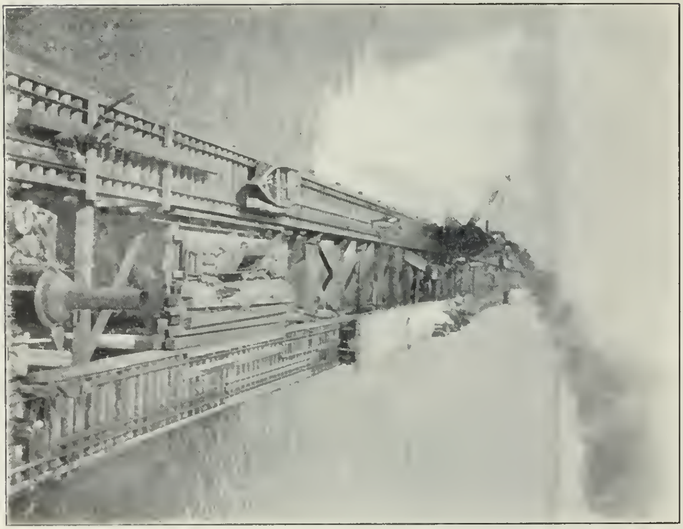


FIG. 1.—TRESTLE, SAN PEDRO BREAKWATER.

FIG. 2.—SLING CHAIN, SAN PEDRO BREAKWATER.

granite from Deelez and Casa Blanca, 80 and 100 miles, respectively, from the work. The sandstone is placed in the lower and inner portions of the substructure. The stone thus far used in the superstructure is granite from Casa Blanca and Deelez.

The stone is loaded on standard flat cars at the quarries, and these cars are run out upon the trestle. The specifications require that, in the construction of the substructure, no stone weighing less than 100 lb. shall be used. At least one-third shall weigh not less than 4 000 lb. Actually there is much 10-ton stone and some 15. The smaller stones, say up to a ton or more, are barred off the flat cars near the axis of the work, much of it in advance of larger stone; while most of the large stones are deposited by cranes, toward the sides of the structure. Thus, while the upper central portion is composed of all sizes over 100 lb. mixed, the lower central portion consists mostly of small stone, and the two sides contain only large stone. Such segregation is not required by the specifications except for the upper portion of the sea slope, but is the result of the contractor's method of construction, and conduces to economy by increasing the voids. From the tonnage and specific gravity of the stone, and carefully sounded cross-sections, the percentage of voids has been computed to be 37½. Under the circumstances this seems rather low, but the dropping of so many heavy stones tamps the mass, and there may be some settlement of the bottom. Besides this, the measured cross-sections are necessarily somewhat too small, for the sounding lead must often slide down into holes.

The cranes used are Barnhart steam shovels with special boom and hoisting tackle substituted for the shovels, and have a lifting capacity of perhaps 30 tons. More than 100 cars of stone have been unloaded in 10 hours by one of these cranes.

The ocean wall of the superstructure is composed of stones weighing not less than 10 000 lb. each, and the harbor wall of stones weighing not less than 6 000 lb. each. Between these walls is a compact filling of stones of all sizes. The top of the superstructure is finished off with stones of large size.

Plates XXVIII, XXIX and XXX show the trestle, the inner and outer faces of the superstructure, the cranes, and the ingenious method of slinging the heavy stones in a chain of two branches which can be released by a tripping device worked from the engine room.

REMARKS.

The new Delaware, Sandy Bay, and San Pedro breakwaters are the largest breakwaters built or building on the seacoast of the United States.

Certain of the breakwaters on the Great Lakes exceed them in length.

A large number of breakwaters of small and moderate size have been constructed on our seacoast. With two exceptions they are rubble-mound breakwaters. The Gloucester breakwater is on the Delaware system, with diminished width and height of superstructure. The Point Judith breakwater is a rubble mound with top and sides above low water, paved with large stones.

Certain of the rubble-stone jetties constructed at bar entrances on our seacoast are comparable in length and quantity of material with our larger breakwaters. For example, the Galveston jetties contain, together, more than 2 000 000 tons of stone.

In plan, all four breakwaters discussed herein are single breakwaters detached from the shore. It has been mentioned that one of the earlier plans for San Pedro contemplated a breakwater attached to the shore, and that the plan under consideration of extending the new Delaware breakwater may lead to a connecting arm between the present work and the shore.

At San Pedro, the ends of the work are to be concrete monoliths, 40 ft. square, 20 ft. high, founded 3 ft. below low water. At Delaware, the ends are rounded, but of the same construction as the remainder of the work.

The Delaware type has stood the test at the mouth of Delaware Bay. At San Pedro no storms of even moderate severity have occurred to test it. At Sandy Bay, where the exposure is undoubtedly greater than at the other localities, the Delaware type has failed.

The Delaware type, particularly as modified for Sandy Bay, is strongly suggestive of the Italian type of a rubble mound faced with blocks in courses. In the latter type concrete blocks are used, and they are often of great size and are carried to great depths, sometimes to as much as 45 ft. below water surface. The Italian system antedates the Delaware type by several years.

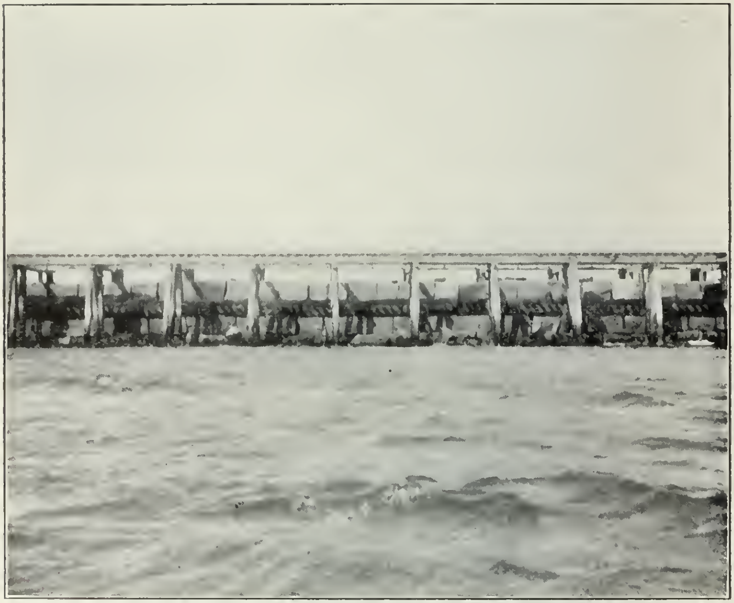


Fig. 1.—Ocean Face of Superstructure, San Pedro Breakwater, 1904.

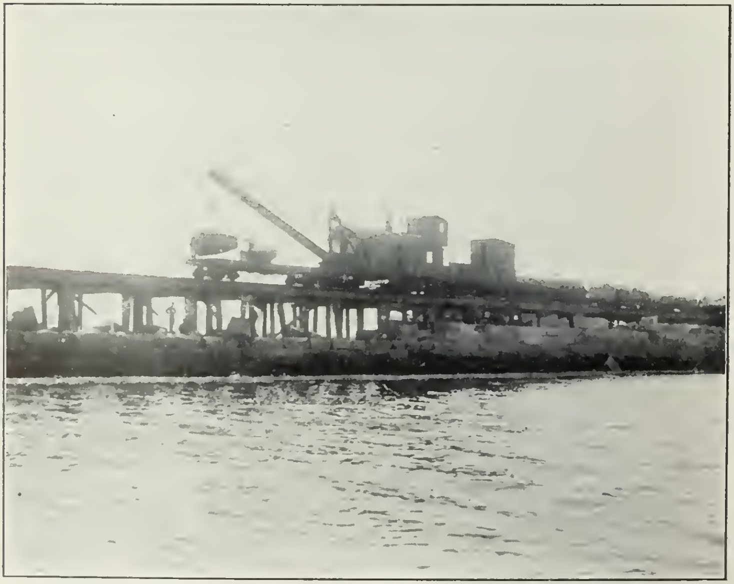


Fig. 2.—Harbor Wall of Superstructure (Incomplete), San Pedro Breakwater, 1904.

There is a striking difference between the new Delaware breakwater as constructed and the cross-section on which the estimate was based. In a depth of 50 ft., the area of cross-section of the substructure as built is 12% in excess of that of the projected cross-section. The advantage of a wide berm of heavy stones on the ocean side is that the top of the substructure, close to the foot of the ocean wall of the superstructure, is protected from the backwash of the waves. Such a berm, however, cannot fail to trip the oncoming waves and cause them to strike a heavier blow against the superstructure.

No necessity for a corresponding addition to the San Pedro breakwater has yet developed. For a time additional stone was deposited along the foot of the ocean wall, but this procedure has been discontinued pending some demonstration of its need.

A berm on the harbor side as constructed at Delaware and San Pedro is a measure of precaution. The harbor slope of the substructure when formed of stone deposited at random depends upon the size of the stone and the height above water from which the pieces are dropped, and slopes that would be stable without a superstructure might not be stable with one added. The blows struck by the waves against the superstructure may cause the resultant of vertical and horizontal forces to pass dangerously close to the inner slope. Again, water which leaps the superstructure and falls close in to the foot of the harbor wall would tend to undermine this wall. At San Pedro a berm, 4 ft. wide, is being built. This supplies a small reserve against slides, but its principal use is to insure a foundation of full width for the bottom course of the harbor wall of the superstructure.

A point of excellence in the present Sandy Bay section is the one-piece capstones of large size. Experience has frequently shown that the first part of a superstructure to suffer damage is the harbor side at the top. In general, a capping of concrete would be cheaper.

The Delaware type of superstructure, by reason of its steep slopes, reduces the volume of substructure required below that required in the purely random type. The cost of such superstructure is, however, much greater per cubic yard than for random work, and whether the Delaware type will be cheaper in first cost for any given place than the random type depends upon the depth of water

and the relative cost of rubble stone and roughly rectangular blocks. In the matter of maintenance, the Delaware type is superior to the random system, unless, in the latter, exceptionally large stones are used. The expense of maintenance at the new Delaware breakwater has so far been practically nothing.

TRANSACTIONS

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INTERNATIONAL ENGINEERING CONGRESS.

1904.

DISCUSSION ON

HARBORS.

By Messrs. L. J. Le Conte, Lewis M. Haupt, W. Henry Hunter, P. W. Meik, H. H. Wadsworth, John H. Darling, J. L. Van Ornum, Clarence Coleman, A. E. Carey, E. L. Corthell, W. Matthews, P. Vedel and Cassius E. Gillette.

L. J. Le Conte, M. Am. Soc. C. E., Oakland, Cal. (By letter.)— Mr. Le Conte
Whether an entrance is to be improved by dredging alone or by twin jetties, a careful study of the sand-drift movements at the site is the first prerequisite. Having determined the prevailing direction of this drift, beyond any reasonable doubt, the engineer is then called upon to decide which is the wisest thing to do, all things considered. In some cases, his judgment will be sorely taxed to decide, but, as a general rule, financial considerations will decide the question one way or the other.

Major Gillette is eminently correct in stating that the "break-out" site is the only proper position for the proposed channel, because it suffers less from choking sand drift at this site than at any other location; and it is most fortunate also that this "break-out" location generally calls for the shortest length of jetties and dredged channel.

There seems to be little doubt that the entrances along the west coast of Florida, like Pensacola, where the drift movement is small, can be most economically improved by dredging operations alone. In making and maintaining these dredged channels, exposed as they are to the incoming sand movement, it is highly important that the dredging plant should so manage and conduct operations as to leave a perfectly free and unobstructed channel to navigation. This means, therefore, that the dredge should dig a navigable channel

Mr. Le Conte. of the full width and depth required; then in addition thereto and closely adjoining the same on the side next to the approaching sand drift, dig another auxiliary and parallel channel sufficiently wide to operate the dredge comfortably and long enough to take in the sand drift. If operations be confined to this auxiliary channel exclusively, the shoaling in the main navigable channel will be practically *nil*. In stormy weather, the dredging plant, of course, would come inside for shelter; meanwhile, the auxiliary channel is intercepting the sand drift. By this arrangement, it is clear that the dredging work would be confined to a limited area and the navigable channel be kept perfectly free to traffic, two very desirable results.

Now that the monthly output of dredges has increased so enormously the unit cost of operations has correspondingly diminished, and the whole financial problem regarding entrance improvement has undergone a marked change in favor of dredging, as being the quickest and most economical method.

Wherever the sand-drift movement is so great annually as to largely overbalance the interest on the cost of twin jetties, then, of course, jetties become the proper thing to build. If the foreshore off the proposed mouth of jetties is steep, with a strong tidal current sweeping across it, then no danger need be feared about a new bar forming outside the end of jetties. On the other hand, should the foreshore be flat for some distance and the deep water sluggish or slack, then trouble is sure to follow in the form of a new bar off the entrance, and the jetties will have to be extended. In figuring out the financial problem as to whether it shall be jetties or dredging alone, it is well to estimate both jetties up to high water throughout. The writer is much pleased to see remarks about the wide gap left near the in-shore ends of the Charleston jetties, which are true in every particular.

While on the subject of theories the writer would like to call attention to certain vague prevalent ideas regarding the volume of sand drifts in motion, so often discussed. The writer has always maintained that the volume of sand drift has been, as a rule, enormously over-estimated in almost every instance, the most remarkable case being that of the Columbia River Bar. Local evidence in the form of sand dunes and other natural features pointed to the probability of there being a prodigious sand movement. As the construction of the jetty progressed, the sand filling followed it out with great rapidity, and when it reached the distance of 4½ miles from Point Adams, a fine channel opened out over the Bar with not less than 31 ft. of water and remained so for seven consecutive years, or until 1902, when Peacock Spit, the true north jetty, washed away, for some unknown reason. The result was that the crest of

Mr. Le Conte.

the Bar increased to 5 miles in length, all 21 and 20-ft. water. This fact alone would amply account for the shoaling up without assuming that the same drift was finally coming round the end of the south jetty in hurtful quantities. The latest surveys, June, 1904, show important changes for the better. The westerly point of the 18-ft. curve of Peacock Spit has advanced straight westerly nearly 1½ miles, thus taking in the No. 0 Buoy. This new shoal is nearly 1 mile wide and is covered by 9 ft. of water, where, in 1902, there were 21 to 22 ft. The south jetty has been extended along the projected line about 5 000 ft. This, together with the sudden enlargement of Peacock Spit on the north, certainly augurs well for a fine channel in the near future. In point of fact, the present distance between the 24-ft. curves, on either side of the Bar, at the channel site, is 1 000 ft. only and nowhere less than 22 ft. at low water. Hence, it is clear that the Bar channel is rapidly cutting out again.

A permanent improvement at this site is not to be expected until the natural north jetty, namely, Peacock Spit, is permanently held up to its work by a suitable stone jetty extended from Cape Disappointment.

As far as the writer knows, there is as yet no positive evidence to show that the sand drift from the south was actually coming around the end of the south jetty, even at this late day, some nine years after the jetty was finished.

As to the proper width to be given between jetties, the very first thing an engineer would naturally seek to assist him in his judgment would be the size and shape of the gorge. It is true, as Major Gillette says, that this section is entirely protected from wave action, but the gorge section represents the true tidal capacity required to fill and empty the harbor with the given tidal prism and with given character of material at the entrance. In combating the sea action outside, the tidal flow due to the tidal prism is all we have to fight with, and the gorge section is the proper one according to the laws of natural hydraulics, as applied to this site. It would appear from this that the width between the jetties ought not to be much more than the width of the gorge.

Mr. Haupt.

Lewis M. Haupt, M. Am. Soc. C. E., Philadelphia, Pa.—The necessity of determining the direction of the resultant drift is fundamental, as stated by Mr. Le Conte and as mentioned in Major Gillette's paper, for it furnishes the key to the successful improvement of ocean bars at much less cost. The difficulties of determining this fundamental question are shown by Major Gillette himself in the several reports he has submitted on this subject. In the final report on the Brunswick Bar,* made in 1901, to Congress, it is stated that the prevailing winds, and the waves and currents generated thereby, drive the channel to leeward. "This is the almost

* House Doc. No. 355, Fifty-sixth Congress, 2d Sess., pp. 3–4.

Mr. Haupt. invariable history of all such bars. * * * The prevailing winds and a predominance of the storms are from the northeast." As a consequence, Major Gillette concluded that if the channel were held in place by a jetty placed to "leeward," so that it could not swing away from this drift, the currents would be forced against the jetty by the advancing sand bank and the channel would be created on the windward side thereof, for the above-mentioned report states:†

"The channel location thus being fixed, Nature may be expected to deepen it, and is assisted in this respect by the fact that the jetty cuts off a considerable escape of the tidal flow, which, added to the water normally flowing through the channel, would be expected to create greater depths. This principle requires the jetty to be located upon that side of the channel toward which the latter is being driven by the drifting sands. To use a phraseology analogous to that which has been used in other cases, it would be on the 'leeward' of the channel.

* * * * * * * * *

"A single jetty on this principle at Brunswick would be located on the south of the channel, since the drifting sands come from the north. * * * At this place * * * the drift * * * is an enormous sand bank which moves and which always moves very positively in one direction."

Major Gillette is, therefore, to be congratulated for the frank change of front which he makes in his present paper where, under the head of "Protection," he says:

"In bar improvement by concentrating jetties, the defence of the channel against drifting sand is a vital element in the problem.

"The jetty on the side from which comes the resultant sand drift is the more important. This has been called the 'windward' jetty. * * * not with reference to the wind, but to the resultant sand drift. Practically it applies to the prevailing wind also, for, as indicated above, cases in which the prevailing on-shore wind and the sand drift do not agree * * * are exceedingly rare."

Again, under "Wind Records," it is said:

"The sand drift being almost invariably with the wind, the record of such winds as cause breakers strong enough to stir up the sand is, perhaps, the best-known index of the sand movement. * * * From a careful study of the above facts, in any particular case, a safe deduction as to the direction of the resultant drift can generally be made."

Yet in the face of these general facts as now accepted by Major Gillette, he cites the single exception of Aransas Pass, where he says:

"The entrance has moved south a long distance against the resultant drift, and work is now going on there, on the theory that the drift is from the north, whereas it appears to be distinctly from the south."

† House Doc. No. 355, Fifty-sixth Congress, 2d Sess., pp. 18-19.

FIG 1.—REACTION BREAKWATER AT ARANSAS PASS.

FIG. 2.—TREE AT ROCKPORT, TEXAS, SHOWING PREVAILING WIND FROM THE SOUTHEAST.

Mr. Haupt.

Thus this vital issue crops out again as it did in the report on the Brunswick Bar, and, as its truth or error affects the economic solution of this class of harbor works, it is worthy of a dispassionate discussion.

Fortunately for Major Gillette's contention, this illustration covers both horns of the dilemma, for if, as he now asserts, the "windward" jetty is the more important one (which is undoubtedly true) and the resultant drift is from the south, then the old Government jetty built out about one mile into the Gulf and on that (the south) side of the entrance, prior to 1885, should have deepened the channel by intercepting the drift on its "windward" side. Unfortunately for the argument, the sand gathered on its northern or leeward side and was dropped in the channel, thus blocking it up, and the bar advanced seaward as fast as the jetty was extended, so that the official reports stated the deepening to have been insignificant and the work was abandoned to private parties. Similar results were produced by building the south jetties first at other Gulf ports.

On the other hand, if the drift is from the northeast and the jetty, now adopted by Congress and nearly completed, is correctly located between its source and the channel, then there should be an improvement, as there is: for in the lee of the completed work the depths are more than 20 ft. at mean low water and with only a portion of the tidal volume under control (see Fig. 1, Plate XXXI). Moreover, if the drift be due to the prevailing winds, it will be found far more reliable to trust to the physical record on the site, which is the result of centuries, than to go to the register of any Bureau and attempt to formulate their observations. Why should this entrance have moved over 2 miles in 40 years "against the resultant drift" when all other inlets move with it? A body does not move toward the force impelling it. But if further evidence be needed as to these effects of the southeast trades at this entrance, a glance at the vegetation in that section, as illustrated in Fig. 2, Plate XXXI, should leave no doubt as to the general direction and intensity of the winds along this coast. This view is looking south-southwest, which is about at right angles to the axis of the foliage, thus indicating a strong east-southeasterly wind. The U. S. Coast Survey Officer, in reporting on this entrance in 1899, said:

"That the resultant current is from the north is shown by the lengthening in former years of St. Josephs Island on the south end, by the accumulation of sand carried from the north and by the washing away of Mustang Island on its north end."

In view of these observations as to the facts from disinterested authorities and especially from the deepening which has resulted from the construction of the work, even during the prevalence of the

Mr. Haupt. summer drift from the southwest, there would seem to be no room for doubt as to the correctness of the theory and location of the breakwater at this point.

It is remarkable, therefore, that on page 310, in discussing the propriety of leaving "a gap" at the shore end to admit the flood tide freely, Major Gillette should state that, although there has been no material change in the floor of the sea in consequence of the currents and waves passing through this breach of 2 000 ft., it is because:

"The tidal range here is small, and the sand drift, while moderate in quantity, is undoubtedly to the north in the aggregate, and seems to be so in detail, seldom or never moving to the south."

And he calls this a "lee jetty," whereas it is, in view of Nature's evidence, a windward breakwater.

The above statement that there is but a moderate quantity of drift and that it is seldom or never moving to the south may, perhaps, best be weighed by quoting Major Gillette's views as given in his Brunswick Bar Report, which devotes large space to the discussion of the work at Aransas Pass, in which this statement occurs:*

"The writer is of the opinion that, while there is a movement at different times in each direction, the great preponderance of the drift is from south to north, and that the jetty is located in direct contradistinction to this primary principle of the theory. The testimony upon this subject is quite conflicting, but a careful analysis will show that the resultant movement is very large and is to the northward."

How are these conflicting statements to be construed in the face of the fact that the inlet has been driven more than 2 miles to the southward within 40 years and has left, in its wake, a train of sand of that length and nearly 1 mile in width? Was it possible for this very great deposit to come up from the south, jump across the channel and settle on the lee side, thus pushing the inlet backward upon itself? If so, it was an unprecedented phenomenon of Nature.

Twin Jetties.—But failing to account for the unprecedentedly good results of the single curved breakwater at Aransas Pass in any other way, Major Gillette, on page 320, states:

"At Aransas Pass, Texas, twin jetties have been partially constructed, the 'lee' jetty being constructed far in advance of the other as the bar was approached. In both cases the drift has passed the line of the windward jetty, which is yet low and incomplete. Very little improvement of the bar has resulted * * *."

To support this assumption of twin jetties at Aransas Pass, Major Gillette stated:†

"The first of these jetties—the north one—is composed of two

* House Doc. No. 355, Fifty-sixth Congress, 2d Sess., p. 24.

† House Doc. No. 355, Fifty-sixth Congress, 2d Sess., p. 28.

Mr. Haupt.

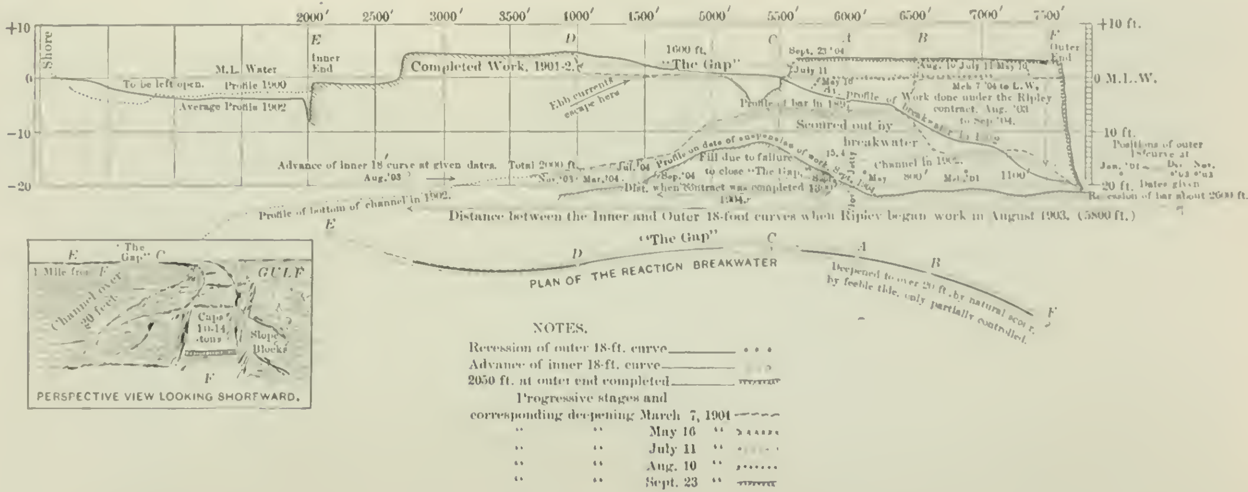


Exhibit showing the "conditions" existing at Aransas Pass, Texas, when the work was suspended under the provisions of the Act appropriating $3 000 000 "for the restoration or maintenance of channels."

Here there has been a remarkable deepening where the breakwater was completed and a shoaling of over 10 ft. where it was not. The bar has been narrowed 80% solely by the action of the breakwater on the currents and the control of the drift.

There would appear to be no other "legal" way to restore and maintain the channel than to close the gap and complete the structure, as enacted by Congress for the purpose of demonstrating the great utility and economy of this system, immediately.

The "conditions," then, would justify the closure of the gap and the opening of a 20 ft. navigation.

Fig. 24.

Mr. Haupt. parts, one of which is a natural bank extending from St. Josephs Island to the inner end of the breakwater."

Again:

"The trend of the currents being such that no artificial structure is needed here to maintain this."

One would naturally infer that there exists a "natural bank" or dike here to obstruct the discharge, yet in the paper now under discussion it is frankly stated:

"A gap at the shore end of this is not even provided with a sill. Apparently it has not scoured appreciably."

The Government plan of two jetties contemplates closing this so-called gap (which is 2 000 ft. long, left as a part of the plan to admit the tides which alone are available for scour) which would, therefore, be a fatal error (see Fig. 24).

To constitute the second jetty, Major Gillette regards as a jetty the head of Mustang Island, which overlaps the breakwater, and the submerged Government work, which, in 1885, or thereabouts, was reported as having "disappeared," being buried under the sand by the cross-currents, the channel having crossed the jetty. It should be noted that, in this paper, this submerged structure is characterized as an efficient jetty, while on page 317, in referring to the 4½-mile jetty, built in large part above the high-water plane, and which served as an effective nucleus for the formation of a sand-spit of the same length, he says: "The work done is not properly a jetty at all." This, notwithstanding the fact that it shifted the channel about 3 miles and caused the gorge as well as the bar to move seaward about 1 mile. And again, on page 316, he also states:

"Under the protection of this jetty, the river promptly built up its south bank, part of it to high water * * *. But the fact that the river cuts a channel with a steep bank, building up its own bank inside the jetty, is due to the protection of the area by the jetty."

One is led to believe, therefore, that it is a jetty after all, and has had a very material effect, as such, in impounding the drift, shifting the channel and advancing the bar without ultimate increase in depth, at the mouth of the Columbia River.

The Nelson jetty, built on the south side of Aransas Pass, was but a repetition of the former experience and met with the same fate, and no effective part of it was left standing at the time Major Gillette cited it as operating as a jetty in co-operation with the breakwater. There is to-day no other structure at that inlet which can be said to form a second jetty, and there has not been since this work was started in 1895.

Bar Advance.—If the projection of the silt forward into the channel can be prevented, there will be no bar advance, and it was

part of the design of the concave breakwater to accomplish this purpose by causing a lateral or transverse movement of the sand to form an automatic spit on the convex bank, which would adjust itself to the regimen of the prevailing conditions. This has been denied by Major Gillette, even in this latest paper, as well as in the Brunswick Bar Report to Congress. Mr. Haupt.

The speaker requested permission to publish an official reply thereto, but it was denied, so that his discussion will be found published in the *Proceedings* of the American Philosophical Society, Vol. XL, for 1901, and need not be dwelt upon here further than to call attention to the progress chart, Fig. 24, as evidence that there has been no bar advance, but a recession of the 3-fathom curve of about ½ mile in less than 1 year, as the work proceeded, and, had it not been suspended by the failure to recommend an allotment from the present emergency bill, the full depth would have been in existence before the publication of this paper.

Cumberland Sound.

Again, while Major Gillette points with just pride to the very satisfactory improvements recently secured at Cumberland Sound, they are due mainly to the rapid building up of the outer end of the windward jetty, thus forming a lee across the crest of the bar. But it is unfortunate that the old trace was followed, since the straight portion of the jetty retires from the effluent current and produces no reaction nor continuous change of direction, so that the scouring and lateral transportation are not fully utilized, and, as a consequence, there has been serious bar advance as shown by the three charts (Figs. 21, 22 and 23) in Major Gillette's paper. In 1900, the 6-fathom curve, abreast of the north channel, was about 12 600 ft. out; in 1902, it was 15 000 ft. out; and, in 1904, it was about 16 500 ft. out, a total advance of nearly ¾ mile in, say, 4 years. This is not so at Aransas Pass, Fig. 24, where, on the contrary, the recession is ½ mile in less than 1 year, and this difference in result is due solely to the curvature which does what it was claimed it would, and to the mode of construction which contemplated the control of the outer currents first. All which features have been frequently submitted to the Boards of Engineers, in connection with proposals for the opening of ocean bars elsewhere with guarantees of depths since 1888, but which have not been made public by them nor reported to Congress.

With the large bank lying between the jetties and the "enormous sand bank which always moves very positively in one direction," it would seem that the windward jetty has not yet completely controlled the littoral drift and must needs be extended. But when it is, it should be flexed to the southeastward, and if the shore end had

Mr. Haupt. been treated differently, so as not to attempt to interfere with the natural disposition of the drift, a permanent bar-crossing could still readily be secured without the need of any part of the south or leeward jetty, the cost of which would be saved.

No human agency can destroy these forces of Nature and permanently prevent the littoral drift from traveling along the coast. The problem is merely so to protect the bar-crossing as to prevent the deposition of sand along its crest, by a reactionary structure which will cause such activity in the currents, locally, that the bottom will be cut out by them and that no other material shall take its place so long as the reaction is maintained, that is, so long as the structure stands. By detaching it from the shore, therefore, there is the least interference with the normal movements of the drift, and the entrance of the tides, while at the same time, the preponderance of the ebb tide is concentrated upon a more limited sector of the bar, which does not advance, but delivers whatever sand passes out to the leeward bank, where the natural forces again pick it up for its journey down the coast to the next breach.*

A Single Windward Jetty.—Major Gillette cites the Columbia Bar as evidence that, even if complete in all its elements, such a jetty will not permanently improve the bar, and attributes the deposit on the channel side to the large quantities of sand brought down by the river, causing "the channel to leave the jetty and wander around the north half of the bar with about the same freedom and the same depth as without the jetty." In short, does this mean that a concentration of one-half has no beneficial effect on the depths? It is also to be noted in the diagram of the crest lines of the bars (Figs. 2 and 3, Plate XXIV) that the crossing-point has moved northwardly some 2 miles between 1895 and 1902, due to the drift of the beach sand coming over from the south, and that it has likewise shoaled from 34 to 24 ft., due to the protrusion of the bar seaward. These are not the ruling depths, however, but an average of a zone ½ mile in width. Then, applying the Aransas analogy, the Columbia jetty must be to leeward of its channel or else the Aransas jetty is a true windward one. Major Gillette records it as his opinion that single windward jetties can never effect a permanent improvement and that it can confidently be expected that the channel will leave the jetty and return to its natural condition. These views are disproven at Aransas Pass, if it be regarded as a "windward jetty," which it has been shown to be, quite as much as that at the Columbia, where the channel has drifted some 2 miles to the northward, in a few years.

If the jetty be detached (see page 318), then it is stated:

* For a fuller discussion of the Cumberland Bar, see *Transactions*, Am. Soc. C. E., Vol. XXXVI, p. 133; Vol. XLVI, p. 507; *Engineering Magazine*, May, 1903, etc.

"The drift will go through the gap (at the shore end) and drive Mr. Haupt. the channel away from the jetty long before the latter has had any chance to help develop a channel by arresting the littoral drift."

But the report* says that it does not do so at Aransas Pass (begun in 1895) and it overlooks the scouring out, not the formation, of the bar coincidently with the construction due to the building up of the outer end of the breakwater, so that the channel has not left "the jetty," which serves as a breakwater and aid to navigation, acting as a visible beacon, on a very easy curve, whose radius is 4 600 ft.

The Columbia River Bar.

Twin Jetties.—The conditions requisite for success are said to be two parallel or convergent jetties "built to high water throughout and extending across the bar from shore," but Major Gillette very pertinently adds, "the material scoured from between the jetties is deposited just seaward of their ends where * * * it is likely to form a new bar."

Applying these generalizations to the mouth of the Columbia River, Plate XXII, independent of the cost, it will be seen that the extension of the windward jetty from Clatsop Spit does not reach even to the crest of the bar and that it is flexed away from the channel-crossing, so as not to support the currents so well as the former single-jetty project which it has supplanted. Moreover, it is found by reference to the Report of the Board, of which Major Gillette was a member,† that the jetties are not to be high-water structures, and still it is stated that:

"No less extensive a project will assuredly secure and permanently maintain a channel, 40 ft. deep and of suitable width."

This claim seems to be at variance with the conditions as above laid down, and there is certainly no known precedent for it in the world. Even to approximate the requirements of a pair of jetties reaching to high water and extending across the bar to deep water would make them cost about $7 500 000, whereas the present estimate is $3 715 000, which is much above the limit fixed by the law ($2 531 140.51). It is proper to add that this plan was the outcome of a proposal to open this bar by natural scour and guarantee the requisite channel at a cost of $2 500 000, but this project, submitted in June, 1902, is still reposing in the archives of the Department. The result of the extension of the south jetty by building out will be a corresponding growth of the sand spit on the south, and, after the north jetty is built, a rolling of the bar farther out, with less depth than at present, because of the longer slope and the ascension of the fresher river water as it passes seaward.

* House Doc. No. 355, Fifty-second Congress, 2d Sess.

† Annual Report, Chief of Engrs., U. S. A., 1903, pt. 3.

Mr. Haupt. The most favorable conditions for channel formation by two jetties which have been properly designed, spaced and built at the mouths of large streams (which are the most favorable locations) are those at Tampico, resulting in 27 ft. at high water; the Sulina Branch of the Danube with 20 ft.; and the South Pass with 30 ft., but this latter has been secured at the expense of the entire pass above.

The topographic conditions at the mouth of the Columbia are well shown in Plate XXXII,* from which it will be seen that a single rocky headland, Pt. Ellice, operating upon only a portion of the stream, has bored, by reaction, to a depth of 100 ft. and transported a portion of the eroded material to the counterscarp, some ½ mile away, while it also operates to maintain a curved channel, having a radius of about 5 miles, for a length of 8 miles. With this natural evidence available, it may safely be predicted that a continuous resisting breakwater, extending a distance of 2½ miles across the bar, having a radius of 5 miles and so placed as to receive practically the whole of the effluent discharge, will cut a channel at least 40 ft. in depth and of ample width, which will be self-maintaining.

The Columbia River problem, therefore, is not a hopeless one, but it is not open to competition and will not be improved by the plans as submitted in the paper, either permanently or otherwise, without radical enlargements and at much greater cost.

The two jetties, two miles apart, cannot co-act to create a channel by scour, but will merely operate as a nozzle to project the ebb currents upon the inner slope of the bar, causing it to advance seaward, while the height is such as to permit a large portion of the sand to travel over in either direction and thus feed the channel and the bar from the littoral drift which is not excluded, thus neutralizing to a large extent the purpose of their construction. In view of the experience already available at this point, at the expense of some twenty years and of over $2 000 000, it would seem hardly justifiable to expend nearly $4 000 000 more to repeat a demonstration the effect of which should be so patent to all maritime engineers and which does not fulfil the requirements which Major Gillette has laid down as necessary to success, as derived from frequent failures elsewhere.

Notwithstanding the many conflicting conditions which Major Gillette mentions as existing, he nevertheless concludes,

"That there are only two methods of bar deepening that are adapted to the conditions that obtain in United States harbors. These are dredging and twin jetties * * *"

It may seem remarkable that the United States is limited to this

* Photographed from model, submitted to the Board of U. S. Engineers to consider and report upon the writer's proposal, but to which report he is denied access.


MOUTH OF THE
COLUMBIA RIVER,
SHOWING LOCATION OF
REACTION BREAKWATER
AND PROPOSED CHANNEL
PACIFIC OCEAN
CAPE DISAPPOINTMENT
OREGON

narrow margin, especially when it is known that other methods have secured permanent deep-water channels, not only in this country, but elsewhere. This is admittedly the most expensive method, not only to construct, but to maintain, requiring more capacious dredging plants than any in existence to-day, and still the bars are not kept under control. Mr. Haupt.

This method also necessitates the additional cost of extending the jetties to keep pace with the bar advance, which Major Gillette calls a "bogey" which has seemingly alarmed many theoretical writers on the subject, yet he introduces this remark by the admission that, "In computing the cost of permanent improvement by twin jetties allowance must be made for the initial scour 'pushing the bar seaward,'" and he follows it by the further admission, "There is no method yet devised, except possibly dredging, that will not 'push the bar seaward' and twin jetties * * * are doubtless the worst sinners in this respect."

It is only necessary to revert to the history of the South Pass jetties to see that the rate of bar advance has been greatly retarded in consequence of their construction, but at the expense of the Pass itself, and, especially, to note the recession of the bar at Aransas Pass, caused by the single reaction jetty to disprove utterly the above assertion that all methods push the bar seaward. Major Gillette evidently cannot appreciate the possibility of a lateral displacement of material by reaction from a concave directrix and the construction by natural agencies of a convex bank at a safe navigable distance therefrom. This is a vital element which has been overlooked in harbor bar problems and its acceptance does not seem to meet with favor as yet. Other channels have been improved by dynamite with dredging, but very few by dredging alone, as is now being undertaken in several instances.

Many other interesting points are suggested in this comprehensive paper, but these may suffice to show that the resources of the harbor engineer are not to be confined to twin jetties and dredging, if the most permanent and economical results are the objects to be attained.

Referring to the paper by Mr. Vedel, the subject of island harbors is an important one. Where there are detached rocks or islands near shore forming a shelter or lee, the area behind them becomes a natural depository for the drift. Some notable instances could be cited, as Richmond Island, Maine, where there is a large rocky island, also at Nantasket, Marblehead and other points in Massachusetts Bay, where similar shoals are formed, connecting with the mainland. Hence, it seems an important point for the artificial protection of a harbor and for the arresting of littoral drift to construct such obstacles at a short distance from the shore. The

Mr. Haupt. distance is an important factor, as illustrated in case of several wrecks near shore, on the New Jersey coast. If too near the low-water line the waves are thrown over and produce erosion of beach, while when they are further out the wave action is checked and deposits take place, causing the sand to advance rapidly to the wreck. Hence it is very important to place the wave breaker some distance off shore rather than close to the beach. Grand Island, Lake Superior, is a notable natural, insular harbor, with comparatively little shoaling, because it has very little drift. Presque Isle on Lake Erie is a case in point. Various other harbors might be mentioned.

Mr. Hunter. W. Henry Hunter, M. Inst. C. E., London, England.—For those who are not familiar with the conditions which form the environment of harbours in the United States, and with the details of the circumstances to which their régime is due, as given by Messrs. Gillette and Gaillard, it is difficult to follow much of the detail. Hence the speaker can only deal in a very general way with the questions that have been raised.

In the discussions which have been read, it is stated with undoubted force, that the first business of the man who is attempting to improve any harbour access is to determine, with accuracy, the direction of the drift of the sand and shingle in the vicinity. That determination is frequently found by experience to be a most difficult matter.

At any rate, so far as the harbours in the islands with which the speaker is most familiar, are concerned, there are two antagonistic theories propounded by British engineers, as to the causes which determine the drift direction. One of these theories is that the drift and its direction is due entirely to wave action, and that it begins to take place at the point at which the wave of oscillation becomes a wave of translation; and the second—which has, in the speaker's judgment, certain facts behind it—is that the direction of the drift is due to the action of the tidal flow, particularly to that of the flood, or governing tide.

It is the speaker's opinion that a combination of the two actions really determines the direction of the drift, that is to say, that the wave of translation serves to render the material more mobile, to disturb the cohesion of the constituent parts of the drift and so to render it amenable to the action of the flood tide which furnishes the transporting force. The activities are, therefore, complex, and the determination of the direction of the drift cannot be regarded as a simple matter—but rather as one requiring accurate and extended observation.

Dealing with the question raised by Professor Haupt as to whether dredging at the entrance to a harbour is, or is not, neces-

sary, in addition to protective works, it seems clear that, from the Professor's point of view, dredging is only a first resource, and that a degree of equation can be arrived at by which natural forces can be directed to the maintenance of the channel, and thus the necessity of dredging be obviated, in illustration of which he advances a very striking and remarkable case. The speaker does not quite understand whether in the case described (where the tidal rise was only 14 in.) there was any assistance obtained from fluvial action. If there is not, in the speaker's opinion, the case is a remarkable one, and if there is any possibility of Professor Haupt supplementing his remarks by a sketch map of the harbour entrance which he describes, such a map will add greatly to the value of the discussion. Mr. Hunter.

In considering the general question as to the necessity for dredging, whether intermittent or continuous, at a harbour entrance, it is necessary to remember, what all know, that the form of the earth's surface is determined by two great series of natural and yet antagonistic operations. The first of these operations, that with which we are most familiar, which can be measured and the effects of which can be anticipated, is of a levelling character. Snow and frost, rain and mist, storm and flood, taken together with chemical action, all operate toward the levelling of the earth's surface, or, in other words, all combine toward a continued transportation of material from a higher to a lower point, the lower point being the bed of the sea and, particularly, the bed of the sea at the coast line in which harbours are indented and through which harbour entrances are maintained. There can, therefore, be no question of equation, which can only be attained when a terrestrial angle of repose has been reached, and harbour engineers have no determined quantity of material to deal with, but a quantity of mobile material which is being added to year by year at an almost incalculable and an altogether incomprehensible rate.

The second of the great series of operations is of a disturbing character and is due, first, to the shrinkage of the earth's crust; and, secondly, to the activity of the internal forces of the earth, of which as yet so little is known to physicists. The effect of this series of operations is that here upheaval is being experienced, there, depression, and thus the levelling effect of the first series is counterbalanced and equation rendered impossible so long as the atmospheric envelope of the globe remains in existence.

Having regard then to the inexhaustible and increasing quantity of mobile material which is available for movement by tidal currents, and to the fact that the velocity (and, consequently, the transporting power) of these currents is greater on flood and ebb, it seems clear that for harbours constructed on foreshores encum-

Mr. Hunter. bered with sand, silt, or mud, and subject to considerable tidal rise, works for the protection and the training of entrances must of necessity be supplemented by serious and persistent dredging operations.

In thickly populated districts, another factor of difficulty is added by the prevalent practice of municipal authorities who pour unceasingly solid matter derived from their sewers and drains into harbours and estuaries which are affected by their proximity. In one particular case, with which the speaker has had to deal, that of the estuary of the River Mersey, some 450 000 cu. yd. of sewage sludge were deposited in the estuary every year. This shews the magnitude of the question, and this and other similar and artificial (and unnecessary) additions to the difficulties of harbour maintenance have all to be considered when the subject of the necessity or otherwise for dredging is under discussion.

With regard to the opinion expressed in one of the papers under discussion, that the volume of sand drift had, in many cases, been overestimated, it can only be stated that in the majority of cases with which engineers are familiar, it would appear that so far from the volume of sand drift having been overestimated, it has been underestimated.

One of these cases is that of the Harbour of Ceara in Brazil, where considerable expenditure has been incurred and the advice sought not of one but of many distinguished engineers, and where, owing to the underestimating of the drift volume after the construction of the works, the last state of the harbour is worse than the first, with the result that the works have been practically abandoned and the harbour left to its fate.

Another case is that of the Harbour of Madras, on the east coast of the great Indian Peninsula, where great works were constructed for the protection of shipping against the monsoon, and where again the sand drift was underestimated, so that it is a serious problem of the present day as to what can be done to maintain even the existing conditions, which certainly have not been improved by the works, so far as depth of water is concerned, though an improvement has been effected with regard to safety during the monsoon period.

Still another case is that at the entrance from the Caribbean Sea to the proposed Nicaraguan line of the Inter-oceanic Canal.

These cases can be multiplied and it is the speaker's opinion that, in most instances, the danger of underestimating the drift volume is much greater than that of overestimating such drift.

The speaker has had little or no practical experience in the construction of island harbours, but has had to consider the matter on one or two occasions. Referring to Mr. Vedel's paper, it appears to him that, in the design of such a harbour, the structure itself must be regarded as presenting an obstruction to the passage of the tidal

wave and of the currents on the coast line, which are due thereto; that it presents a case similar to that of the pier of a bridge on an enormously enlarged scale and that, therefore, it is necessary in the design of the harbour to have regard to that fact. Mr. Hunter

No modern bridge designer would dream of suggesting that a pier of a bridge should, so far as the portion below the level of the water is concerned, be constructed in the form of a parallelogram, but would endeavour to minimise, by the form of the pier, the obstruction caused by it to fluvial currents; and the principle which is applied to the minor obstruction certainly should be applied to the major obstruction, *i. e.,* the island harbour.

The difficulty which has so frequently arisen in the formation of lines of communication between the island harbour and the shore, when the harbour is required for use for traffic purposes as well as for protection, might find a solution in the utilization of reinforced concrete, "ferro-concrete," or "concrete-steel," which is coming into such widespread use and of which connecting viaducts might efficiently and economically be constructed.

P. W. Meik, M. Inst. C. E., London, England.—The speaker agrees to a very great extent with the last speaker, Mr. Hunter, in what he has said with reference to the difficulty of discussing the papers of Majors Gillette and Gaillard in detail, and he feels, therefore, also compelled to limit his remarks to general features and general principles. Mr. Meik.

It may be taken for granted that no scientific theory can be accurately built up except after a large accumulation of experimental knowledge. One of the greatest mistakes that a scientific man can make is to propound a large theory on an isolated fact or a limited number of circumstances. In connection with harbour engineering, with which the speaker has had some little experience, it is a fact that the data are too limited to make any general laws which could be usefully applied in the designing of harbours of refuge or protective works on the sea-coast. On one or two branches of the question, however, we can go to a certain length. Mr. Hunter has referred to one of these, and that is with reference to the question of littoral drift. Major Gillette's paper seems tolerably sound in not neglecting to emphasise the importance of wave action. In Mr. Matthews' paper, the same subject is referred to, and he also emphasises the importance of wave action on littoral drift, but the speaker does not agree with what he says, to the effect that there are two schools on the other side of the water which take different views on the subject. He thinks the number of men in the profession, who emphasise more particularly the action of tidal and other currents in the movement of the sea bed, is so very small that it may be taken for granted that the preponderating opinion among British engineers is, that by far the major part of the movement of the

Mr. Meik. material forming beaches and the bed of the sea is due to wave action, and there has never been any doubt on the subject. Evidence of it is forthcoming at every step one takes in harbour work. Mr. Matthews quotes instances in support of this view and it is not necessary to give further cases. It should not be assumed, however, that those who support the theory of the importance of wave action on littoral drift mean to say that there is no action at all by current, but merely that when one comes to deal with harbours on a sea-coast, the action of the current is so small that the wave action is the important factor to be taken into account.

The speaker wishes to add one other remark with reference to something Mr. Hunter has said concerning the difference of waves of translation and waves of oscillation. In any works, which the speaker has read on the subject, it has been assumed that the two classes of waves are entirely distinct in character, and that a wave must be either one or the other. In his opinion, this is not so. In the wave of oscillation, the water travels in an ellipse in which the major axis is vertical. In the wave of translation, the particles of water also travel in an elliptical form until the wave is broken, but the major axis is horizontal. The speaker thinks there is no absolute line of demarkation between the two waves, but that as the oscillatory wave passes into shallower water the horizontal axis of the ellipse increases while the vertical axis shortens until the wave partakes entirely of the "translation" character. Where deep water continues close in to the shore there may be a wave which has so much of the character of the wave of oscillation that it may exert comparatively little horizontal action against a pier or breakwater. Hence the observed height of the waves at any harbour is not of itself any clear guide as to the force of the sea to be provided against.

Mr. Wadsworth. H. H. Wadsworth, M. Am. Soc. C. E., Superior, Wis. (By letter.)—The features of the Duluth-Superior Harbor of most interest to engineers are, doubtless, the piers at the two entrances, and these stand as magnificent monuments to their builders, but the item of greatest importance in the improvement of the harbor, both in the matter of cost and in results, is the dredging, which has produced channels and basins of such dimensions that the largest vessels on the Lakes (one of them, 560 ft. in length, carrying cargoes of from 10 000 to 12 000 tons) regularly enter, proceed from dock to dock and leave, entirely unaided by tugs.

This work upon which about two-thirds of the $3 000 000 appropriation has been expended may be of sufficient interest to warrant supplementing Major Gaillard's paper with a few additional facts. Table 21 shows the quantities dredged each season during which the contracts were in force, by each dredge used on the work, together with its total time on the work, not counting Sundays and holidays:

Mr. Wadsworth.

TABLE 21.

DREDGE.	1897.		1898.		1899.		1900.		1901.		1902.		Total.
Number or name.	Time, in days.	Cubic yards.	Time, in days.	Cubic yards.	Time, in days.	Cubic yards.	Time, in days.	Cubic yards.	Time, in days.	Cubic yards.	Time, in days.	Cubic yards.	Cubic yards.
No. 1	135	220 990	193	312 688	140	214 705	8	11 776	....	....	....	....	760 159
No. 2	41	33 402	169	183 597	48	132 271	161	451 290	176	561 088	186	693 182	2 055 130
No. 3	135	158 321	162	333 733	175	440 019	172	398 134	137	326 162	169	405 163	2 061 532
No. 4	146	207 921	190	279 593	177	287 475	9	14 970	13	14 263	....	....	804 222
No. 7	113	149 382	115	256 066	183	587 679	165	588 167	37	88 382	....	....	1 670 276
No. 8	139	139 669	144	308 109	164	451 018	167	455 020	167	417 472	199	570 602	2 341 890
Napoleon	....	....	....	....	....	....	....	....	117	463 690	....	....	463 690
Milwaukee	114	239 537	190	408 494	184	377 454	103	248 326	90	161 047	71	144 459	1 579 317
Menominee	44	91 514	190	402 494	183	402 611	173	392 544	158	340 850	158	393 998	2 024 011
Portage	119	207 163	186	326 663	177	367 294	173	294 730	141	278 944	168	423 953	1 893 747
Superior	....	....	175	328 618	179	287 706	98	174 672	174	296 498	62	115 727	1 298 221
Port Huron	....	....	163	281 698	87	197 908	76	149 110	58	73 201	95	254 681	956 598
West Superior	....	542	145	756 885	155	977 871	166	749 555	181	853 311	118	383 332	3 721 496
Northwestern	71	150 843	....	....	....	....	....	....	....	....	....	....	150 843
Clam-shell	....	11 081	....	....	....	....	....	....	....	....	....	....	11 081
Total		1 610 395		4 178 638		4 724 011		3 928 294		3 870 508		3 385 397	21 697 213

Note: The first twelve of the above-mentioned dredges were dipper-dredges and the next two were hydraulic dredges. For the hydraulic dredges, *West Superior* and *Northwestern*, 1 day = 24 hours; for all other dredges, 1 day's work was 16 hours.

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This table shows the great difference in the capacities of the various machines, some of them being old and small and designed for much less depth of digging, while others were modern dredges in all respects.

No averages of daily quantities dredged by individual machines during the whole period of the contracts are given. Some of the dredges were entirely rebuilt between dredging seasons, so that an average would be no indication of the capacity of a dredge, either before or after rebuilding. This fleet of dredges was attended at times by as many as 14 tugs and 27 dump scows, the capacities of the latter ranging from about 200 to over 600 cu. yd.

The disposition of the dredged material was as follows:

Dumped in Lake Superior	14 841 141 cu. yd.
Dumped at docks in process of construction and rehandled for filling	1 269 852 " "
Discharged by hydraulic dredges on spoil banks	3 882 339 " "
Dumped in Spirit Lake and in abandoned portions of old St. Louis River channel, inside harbor lines	632 956 " "
Dumped inside harbor lines in Superior and St. Louis Bays, on account of storms too severe for dumping in Lake Superior	1 070 955 " "
Total	21 697 243 cu. yd.

The harbor was divided into three districts. The quantity dredged in each was as follows:

District No. 1	12 239 382 cu. yd.
" No. 2	5 168 051 " "
" No. 3	4 289 810 " "
Total	21 697 243 " "

District No. 1 comprised Superior and Allouez Bays, with Duluth Canal and Superior Entry (see Fig. 1, Plate XX). The price per yard was 7½ cents. A large portion of the work was near the entrances to the harbor, so that the tow to the lake was not a long one. The material was mostly sand, though some mud was encountered in several places, and, in the channel along the harbor line between the letters, *R* and *I*, in the word Superior in Superior Bay (see Fig. 1, Plate XX), there was a deposit of boulders, of all sizes up to 4 cu. yd., about 1 000 ft. in length. This was all removed at the same unit price. Work, however, was not crowded on this section, but extended over the whole six years. Adjoining this

Mr. Wadsworth.

boulder deposit on the southeast is a section about 1 mile in length, and a little more than ½ mile farther southeast is another, both of which sections were taken out mostly by hydraulic dredging.

The two long narrow islands and the long point of land shown on the map (Fig. 1, Plate XX) just inside the harbor lines opposite these sections are spoil banks.

In District No. 2, comprising St. Louis Bay, the price was 8 cents per yd. The material was mostly mud and clay and was nearly all towed to Lake Superior, through the Duluth Canal, an average haul of about 5 miles.

In District No. 3, extending from the railway bridge crossing the river just below Grassy Point to Spirit Lake, the price was 10 cents per yd. The material varied from a soft clay, through a very fine silicious mud to moderately coarse sand.

The portion of this district lying east of Grassy Point was dug by dipper-dredges, and the material towed to the lake. Above Grassy Point, the old natural channel, which was very crooked, was abandoned entirely and a new channel, running nearly due west about 1½ miles, thence by a curve of slightly more than 6 000 ft. radius for nearly 2 miles, followed by about ½ mile of curve of 1 500 ft. radius to join the old channel at the foot of Spirit Lake, was dredged 200 ft. in width, increased to 260 ft. on curves. This channel was opened its entire length, and a large portion of the material was removed by the hydraulic dredge, *West Superior*.

The long narrow islands shown north and west of this channel and some not shown on the map (Fig. 1, Plate XX) are spoil banks. During the season of 1902, some of the dipper-dredged material was dumped into the abandoned reaches of the old channel, but a considerable portion was towed 7½ miles to Lake Superior.

The contracts provided that work done by hydraulic dredges should be measured in the cut, and that 5% should be added to place measurement, the dipper-dredged material being measured in scows.

A system of keeping account of the work done by a hydraulic dredge was developed, which enabled the dredge inspectors to keep their record of the output up to date. This consisted simply of cross-sectioning the cut at regular stations (soundings taken at stern of dredge) and comparing the soundings with the original ones, or with those taken on previous cuts at the same stations. For this purpose, the stations were marked by cross-ranges, or were located by measurements from fixed points. During the season of 1898, monthly estimates of hydraulic work were made up from soundings, covering all the work done during the month, taken from a boat and located by transit observations. During the following season, this method was used a few times to check the inspectors' results. Subsequently, the estimates made from daily reports of inspectors

Mr. Wadsworth. were checked, by independent soundings, only at the close of the season, when they were generally found to be not more than 2% in error.

The curve of 6 000 ft. radius, already mentioned, was laid out in 500-ft. chords. Piles were driven around outside of the curve at the extremities of the chords. Radial ranges were established at each of these points; and then, for use in locating line ranges for dredges, the distances from each of several successive piles to the desired lines were computed and tabulated.

In general, it is expected that the widths and depths of channels and basins will be maintained with very little further dredging, and that only at long intervals. The St. Louis River, above Fond-du-Lac, flows over a rocky bed and carries no silt. Just what effect the strong current of this stream in times of floods will have on the rectified channel from Spirit Lake to Grassy Point is as yet undetermined.

The Nemadji River (the stream entering Superior Harbor Basin, shown near the bottom of Fig. 1, Plate XX, flows, for the lower 20 miles of its course, through a red clay formation, and carries, during floods, a great amount of material in suspension. The quantity deposited in the dredged channel and basin in the vicinity of the mouth of this stream during the season of 1903 exceeded 100 000 cu. yd. How much this deposit will average per year is not yet known, but it will evidently be enough to require some dredging as often as every second year.

The force of inspectors, as organized for superintending this work, was very complete, consisting of two chief inspectors, one in charge of each of the two contracts, who were each provided with a gasoline launch, enabling them to make daily visits to all dredges in their charge. They laid out all work and kept a constant check on the work of the dredge inspectors. They made weekly and monthly reports of progress, the latter containing progress maps. Of dredge inspectors, there was one for each 8-hour daily shift on each dredge, that is, two inspectors for a dipper-dredge working 16 hours daily and three for an hydraulic dredge working 24 hours per day.

The dredge inspectors looked after the maintaining of buoys and stakes on ranges and the position of the dredge on these ranges, sounded the cut being dredged carefully and frequently, saw that proper depths were adhered to and made careful estimates of each scowload, keeping a record of the overload and underload on each scow (the capacities being calculated for an even full load). A careful record was also kept of the time of departure of scows from the dredge, of time lost from actual work and the cause of same, in fact a complete record of each day's proceedings.

Dumping inspectors were stationed at the dumping ground so that the disposition of every scowload of dredged material was known, and a check was had on the estimates of dredge inspectors. Mr. Wadsworth.

At the time these contracts were let, February, 1897, prices, both for labor and material of all kinds, were very low. A marked rise occurred during the following year, so that the contract prices were insufficient to yield a substantial profit to the contractors. It is much to their credit that the work was pushed energetically and completed within the specified time.

JOHN H. DARLING, M. AM. SOC. C. E., Duluth, Minn.—Having been connected with the work of harbor improvements on Lake Superior for the last twenty years, the speaker can certify to the general correctness of Major Gaillard's statements, and is pleased to see so comprehensive and fair an exposition of the engineering and commercial features of that great lake. Mr. Darling.

In Plate XXII, containing cross-sections of various piers and breakwaters, Figs. 1 and 2 deserve special consideration; Fig. 1 being a good example of combined timber and concrete construction, and Fig. 2 being a novel type of all-concrete construction, which is now being built at Superior, and which may prove to be an advantageous and useful form. The author states:

"The submerged weight of such a crib (that of Fig. 1) is about 50 lb. per cu. ft., while the submerged weight of concrete is 87 lb. per cu. ft. It is therefore evident that for equal stability against overturning or against displacement by waves, the area of cross-section of a concrete pier or breakwater would be considerably smaller than in the case of one of crib work."

An investigation of the relative stability of these two forms of piers brings out these facts:

Case 1.—Taking two substructures alone, one of crib and the other of concrete (Figs. 25 and 26), having net weights, immersed in

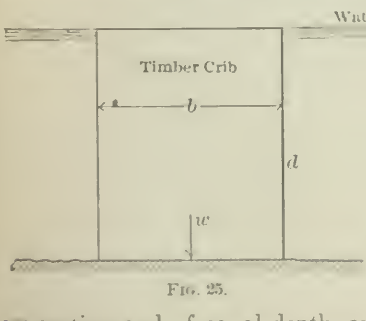


FIG. 25.

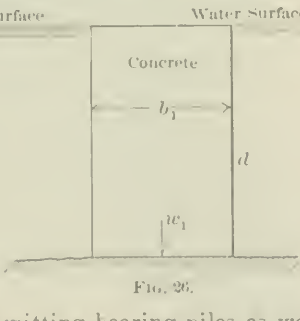


FIG. 26.

cross-section and of equal depth, and omitting bearing piles as well as superstructure, the thickness of the second would have to be 76% of the first for equal stability against overturning.

Mr. Darling. *Case 2.*—If a concrete superstructure is added to each of the foregoing substructures, of solid rectangular section (Figs. 27 and 28), each having the same thickness as the substructure, and a height such that the relative volumes of superstructure and substructure are the same as in Fig. 2, Plate XXII, then for equal stability, as to overturning, the second one, which is entirely of concrete, would require a thickness of 85% of the one with a timber substructure.

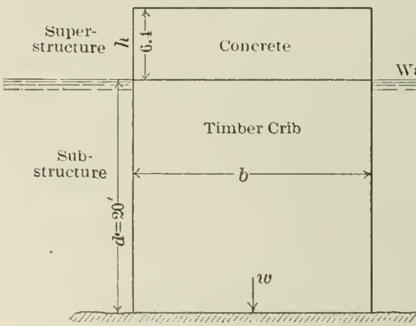


FIG. 27.

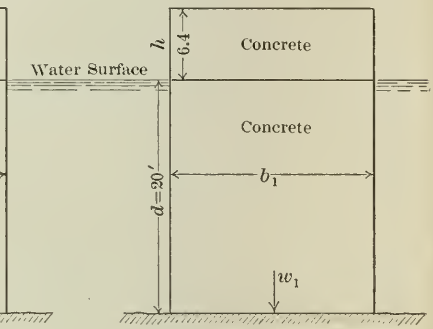


FIG. 28.

Case 3.—If bearing piles are added to the last-mentioned two forms (Figs. 31 and 32), similar or equivalent to the bearing piles shown in Figs. 1 and 2, Plate XXII, then the thickness of the second form probably need not exceed 78% of the former.

The method of determining the foregoing results is by computing the moments of weight with reference to one edge of the base. Represent by W the weight of a linear foot of structure, by C, c and t the net weight of a cubic foot of concrete in air, of concrete immersed in water, and of a timber crib immersed, taken as 150, 87, and 50 lb., respectively.

Under Case 1, the two blocks are shown in Figs. 25 and 26.

The moments of resistance to overturning, denoted by M and M_1, are:

$$M = W \frac{1}{2} b = b\, d\, t \frac{1}{2} b = \frac{1}{2} d\, t\, b^2 \qquad (1)$$

$$M_1 = W_1 \frac{1}{2} b_1 = b_1 d\, c \frac{1}{2} b_1 = \frac{1}{2} d\, c\, b_1^2 \qquad (2)$$

For $M = M_1$ we have $t\, b^2 = c\, b_1^2$ and $b_1^2 = \frac{t}{c} b^2$

$$\text{and } b_1 = \sqrt{\frac{t}{c}}\, b = \sqrt{\frac{50}{87}}\, b = 0.76\, b \qquad (3)$$

as first stated.

Under Case 2, the two blocks are shown by Figs. 27 and 28. The depth of substructure, $d = 20$ ft., is that of Fig. 2, Plate XXII, and

the height of superstructure, $h = 6.4$ ft., is that of the superstruc- Mr. Darling. ture of Fig. 2, Plate XXII, if it were reduced to a solid rectangle having the same thickness as the mean thickness of the substructure of Fig. 2, Plate XXII.

Then $M = W \frac{1}{2} b = (b\,h\,C + b\,d\,t) \frac{1}{2} b = \frac{1}{2} (h\,C + d\,t)\, b^2 \ldots . (4)$

$$M_1 = W_1 \frac{1}{2} b_1 = (b_1\,h\,C + b_1\,d\,c) \frac{1}{2} b_1 = \frac{1}{2} (h\,C + d\,c)\, b_1^2. \quad (5)$$

For $M = M_1$

$$b_1^2 = \frac{h\,C + d\,t}{h\,C + d\,c} b^2 = \frac{1\,960}{2\,700} b^2 \text{ and } b_1 = 0.85\, b \ldots\ldots\ldots \quad (6)$$

as previously stated.

Referring to Case 3, an attempt will be made to determine the relative stability, with respect to overturning, of the forms shown in Figs. 1 and 2, Plate XXII, and to express the same in the form of a ratio between the thickness of the two forms for equal stability.

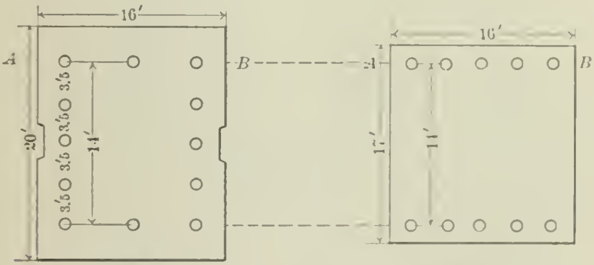


Fig. 29. Fig. 30.

In order to avoid too great complication, certain modifications are introduced.

a.—The cross-section of the substructure and of the superstructure of Fig. 2, Plate XXII, is reduced to equivalent rectangular figures, shown in Fig. 32 by full lines, and where the outline of Fig. 2, Plate XXII, is shown by a dotted line. This apparently introduces an error by narrowing the base and thus diminishing the stability. But it is assumed in the computation that the turning point is at the extreme edge of the base, A, as if resting on an unyielding bottom, as of rock, whereas the actual bottom is usually a yielding material, it being sand at the site of the pier under construction, so that the effective point of overturning would be at some distance inside the edge of the battered face, A_1.

b.—The water surface is brought down to the top of the substructure, instead of 1 ft. above it.

Mr. Darling. *c.*—The depth of substructure in Fig. 1, Plate XXII, is reduced by 1 ft. in Fig. 31 to make it equal with that of Fig. 2, Plate XXII, or Fig. 32, and the superstructure in Fig. 31 is made of equal height with the superstructure in Fig. 32. It happens that the concrete in Fig. 1, Plate XXII, is equivalent to a rectangle having a base equal to the thickness of the superstructure and a height of 6.33, which is very nearly the same as drawn in Figs. 31 and 32.

d.—The arrangement of bearing piles is also altered for the purpose of simplifying the comparison, but in such a manner as to maintain an equivalent resistance to overturning.

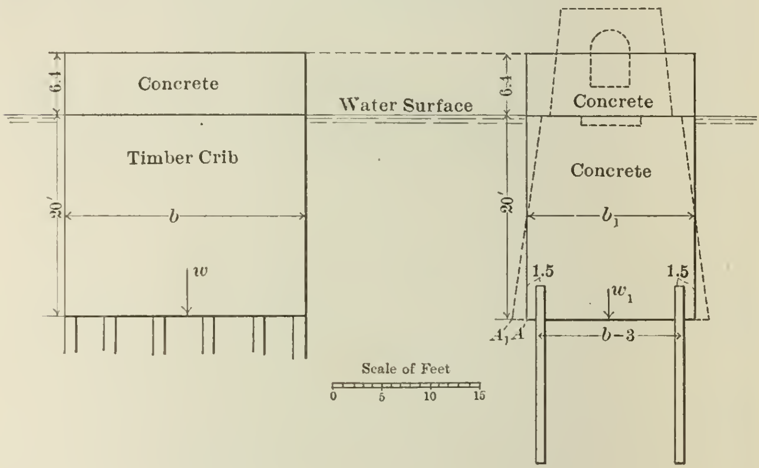


FIG. 31. FIG. 32.

Fig. 29 shows the distribution of piles under one of the monoliths which is 20 ft. wide and 16 ft. long, the small circles indicating piles. Fig. 30 shows an arrangement, which may be called theoretical, for the special purpose of this investigation. The action of the piles is here considered independently of whatever support is afforded by the earth between and around the piles, and also independently of the weight of the blocks. Suppose the force acting upon the pier is from the front, tending to overturn about the piles on the line, *A B*, Figs. 29 and 30. The piles, *A*—*B*, are supposed to be absolutely unyielding from compression, and the remaining piles are supposed to be in tension and to yield by slipping or withdrawing from the concrete, when sufficient force is applied. It is assumed that the resistance of the several piles is proportional to their distance from the line, *A B*. This assumption may be open to question, but it is believed to be an approximation to the truth. Let $f =$ ultimate resistance of a pile to withdrawal from the con-

crete, $y_1 =$ distance of the farthest pile from the axis, $A\ B$ (=14 ft. in Fig. 29.) Mr Darling

p = stress on any one pile, and y the distance of that pile from $A\ B$. $\frac{p}{y}$ is constant for all the piles and equal to $\frac{f}{y_1}$. The stress on any pile is $\frac{p}{y}\ y = \frac{f}{y_1}\ y$, and the moment of resistance is $\frac{f}{y_1}\ y^2$. The total moment of resistance is the sum for the several piles, or $M_o = \frac{f}{y_1}\ \Sigma\ y^2$. In Fig. 29, $\Sigma\ y^2$ is found to be 930, and y_1 is 14 ft., so that, $M_o = \frac{f}{14}\ 930$.

To find the number of piles when arranged in two rows, 14 ft. apart, as in Fig. 30, which will have the same amount of resistance as those in Fig. 29, divide M_0 by 14 to give the stress on the rear row, and further divide by f to give the number of piles in each row, making $\frac{930}{14 \times 14} = 4.74$ piles, or 9.48 piles in all for the block. In Fig. 30, the number is taken as 10, to avoid fractions. The number of piles per linear foot is $\frac{10}{18}$, and the number of pairs of piles, consisting of one in front and one in rear, is $\frac{5}{18}$.

As to the ultimate resistance of a pile, exact data are lacking. They are driven, with the aid of a water jet, 15 ft. into the ground, the large end down, in sand, and the resistance to withdrawal must be great; from the writer's observations of attempts to draw piles, it might be 50 tons. The resistance to withdrawal from concrete, when the small end is embedded 3 ft., as in Fig. 2, Plate XXII, would be less, but would likely be as much as 10 tons, although this is a mere guess. It would, of course, be greater, if the piles reached farther into the concrete, or if they were notched on the sides. Calling the resistance 10 tons, gives for the moment of resistance of one pair, $10 \times 2\,000\ (b_1 - 3)$, as may be seen by an inspection of Fig. 32 and taking 1 ton as 2 000 lb.; and the moment for 1 lin. ft., $\frac{5}{18} \times 10 \times 2\,000\ (b_1 - 3) = 6\,250\ b_1 - 18\,750$. This moment forms the last two terms of the following Equation 8.

From Figs. 31 and 32, we write equations of moments similar to Equations 4 and 5.

For Fig. 31. $M = \frac{1}{2}\ (h\ C + d\ t)\ b^2$ ....(7)

For Fig. 32. $M_1 = \frac{1}{2}\ (h\ C + d\ c)\ b_1^2 + 6\,250\ b_1 - 18\,750$ ......(8)

Equating 7 and 8, and introducing the previously adopted values, $h = 6.4$ ft., $d = 20$ ft., $C = 150$ lb., $c = 87$ lb., $t = 50$ lb., we obtain

$$b_1^2 + 4.6\ b_1 = 0.73\ b^2 + 13.9 \quad \text{(9)}$$

and

$$b_1 = -2.3 \pm \sqrt{0.73\ b^2 + 19.2} \quad \text{(10)}$$

Mr. Darling. If $b = 24$ ft., as in Fig. 1, Plate XXII, and rejecting the negative root of Equation 10

$$b_1 = 18.7 \text{ ft., or } 78\% \text{ of } b.$$

This value of b_1 does not differ greatly from that of the author's cross-section in Fig. 2, Plate XXII (the mean thickness of the substructure), which is 17 ft.

In what precedes, the holding power of the pile in the concrete is taken as 10 tons. By increasing the penetration in the concrete, or by cutting notches in the pile for the concrete to enter, the resistance would be greatly increased. For example, if one of the piles shown in Fig. 2, Plate XXII, were girdled with a dap, as at A in Fig. 33, 1 in. deep and long enough to prevent shearing of the concrete, and 12 in. from the top of the pile, and taking the pile as 12 in. in diameter, the shearing surface at B, indicated by the dotted lines, would be 377 sq. in. Taking the resistance to shearing as 200 lb. per sq. in., which is probably low enough for any timber used, which is water-soaked, the total shearing resistance would be 37 tons. If 37 tons instead of 10 tons were used in Equations 8 and 10, it would give, for $b = 24$ ft., $b_1 =$ 14.8 ft., or 62% of b.

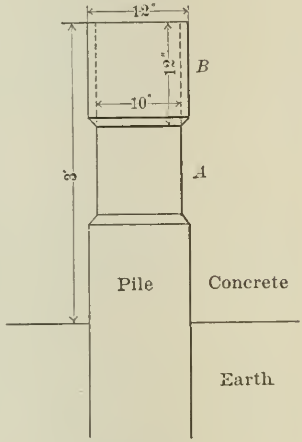


FIG. 33.

Instead of such a notch as that shown in Fig. 33, several smaller notches might be made more easily, and be equally effective.

Major Gaillard in designing the Superior Entry piers fixed upon the dimensions of the cross-section shown in Fig. 2, Plate XXII, after a consideration of certain actual forces tending to overturn the structure, together with the resistance of the structure to overturning, computing the moment of each force in foot-pounds. The purpose of this discussion is, as stated, limited to a comparison of the two forms, Figs. 1 and 2, Plate XXII. These being reduced to equal vertical dimensions, the forces tending to overturn do not enter into the argument, and the resisting moments are expressed in general terms. The comparison, however, establishes relations which are believed to be correct, except as affected by the uncertainties in some of the constants, and tends to confirm, at least approximately, the stability of the cross-section given in Fig. 2, Plate XXII, by an independent line of reasoning, based upon the form shown in Fig. 1, Plate XXII, taken as a standard, the stability of which is now established by its four years' existence.

Relative Cost of Types of Pier Shown in Figs. 1 and 2, Plate XXII.—Assuming that both these forms are substantial and effective for the purpose intended, the relative cost of construction is a matter of practical importance. Major Gaillard states that the Entry piers Mr. Darling.

"are estimated to cost less per linear foot than if built of rock-filled timber cribs of equal stability surmounted by a concrete superstructure." (See Fig. 2, Plate XXII.)

This is not yet fully determined, as the most exposed portions of the Entry piers remain to be built. There are several reasons tending to make the cost of the Entry piers less than that of the Duluth piers, aside from the thinner section made possible by the use of subaqueous concrete and embedded piles, which will be mentioned here.

1.—The average depth of the Entry piers is 2.1 ft. less than the Duluth piers.

2.—The pierheads at Duluth form a larger percentage of the entire length of the piers than at the Entry; the Duluth piers having 3 414 lin. ft. and the Entry piers 6 464 lin. ft., while the large pierheads have approximately the same length in each case.

There is another consideration in addition to that of stability, namely, the width required for other purposes than for simply serving as a jetty to confine the channel and prevent the littoral movement of sand, or to serve as a retaining wall. At Duluth, the piers are much used as a promenade. Being near the center of the city, and attractive in appearance, they are visited by large numbers of people who go down to see the piers and the waters of the Canal and Lake and to watch the vessels passing through. The Entry piers (Fig. 2, Plate XXII), of narrower width and without parapets or railings would not serve this purpose as well, and, for this reason, would be less suitable for such a locality as Duluth, but may answer for the Superior Entry which is less accessible to the public.

The cost of the two forms of structure may be expressed in the following manner, which will permit of an approximate comparison. Suppose the two forms to be represented by Figs. 31 and 32, which are intended to be equivalents of Figs. 1 and 2, Plate XXII, as already stated.

Let a' = cost of timber crib per cubic foot.
x' = " " concrete " " "

Then the cost of Fig. 31 per linear foot is

$20\,b\,a' + 6.4\,bx'$ + cost of bearing piles. . . . . (11)

The cost of Fig. 32 per lin. ft. is likewise

$20\,b_1x' + 6.4\,b_1x'$ + cost of bearing piles . . . . (12)

The cost of the bearing piles will here be considered the same for

Mr. Darling. both figures.* Also put $\frac{b_1}{b} = r$, that is, the ratio of thickness in Figs. 31 and 32 for equal stability (the value of which may be 0.76 or less), and $b_1 = r\,b$.

If we place Equations 11 and 12 equal to each other and solve for x', we will obtain a value for x' (the cost of concrete), such as would make the cost of the two forms equal. Then it would remain to be seen whether the actual cost of the concrete was less than x', and how much. Thus, by equating 11 and 12, we find

$$x' = \frac{20\ a'}{26.4\ r - 6.4} \quad \ldots\ldots\ldots\ldots\ldots\ldots (13)$$

In order to change from cubic feet to cubic yards let

a = cost of timber crib per cubic yard.
x = " concrete " "

Then $x' = \frac{x}{27}$, and $a' = \frac{a}{27}$, and these substituted in 13 give

$$x = \frac{20\ a}{26.4\ r - 6.4} \quad \ldots\ldots\ldots\ldots\ldots\ldots (14)$$

For $r = 0.76$, as previously deduced in an approximate manner,

$$x = 1.47\ a \ldots\ldots\ldots\ldots\ldots\ldots (15)$$

giving a cost for concrete about one and one-half times that of crib-work in order to make the cost of the two types the same per linear foot.

As to the value of *a*, Major Gaillard states:

"Including the pile and stone foundation and steel armor, the cribs at the Duluth Canal cost about $5 per cu. yd. in place."

Deducting the cost of the piles, which amounts to about 62 cents per cu. yd. of the crib volume, and the cost of a stone embankment which in places reached below the base of the rock shown in Fig. 1, Plate XXII, which amounts to about 15 cents per yd. of crib volume, reduces the cost to $4.23.

The cost of cribwork elsewhere in the Duluth District has been less. For example, on the Marquette breakwater the contract price, in 1892, amounted to $2.92 when reduced to a cubic yard; at Marquette, in 1894, it was $2.64; at Agate Bay, in 1898, $2.34; and at Grand Marais, Minn., in 1900, $3.49. Increasing the first three of the four just named by 52 cents, to cover the increase of $7 per M. in the cost of timber, and adding 10% for administration, the mean of the four is $3.52 per cu. yd. for the probable cost at the present time of rock-filled cribwork, such as shown in Figs. 5 and 10, Plate XXII.

* There is evidently an excess of piles in Fig. 1, Plate XXII (and Fig. 31). The load per pile is only 15½ tons (Annual Report, Chief of Engrs., U. S. A., 1901, Appendix KK, p. 2 861), the load per square inch on pilehead is 205 lb., the compression of timber is found not to be great, and quite uniform, and might safely be exceeded. It is thought that one half the number would be sufficient. This would give an equal number of piles per linear foot, and very nearly an equal loading for each pile, as in Fig. 2, Plate XXII.

As to Fig. 1, Plate XXII, it is now believed that a considerable reduction could be made in the cost, without undue sacrifice of strength, by modifying certain details and by omitting some of the accessory works that were found to be unnecessary, so that a suitable crib could be built for, say, $4.00 per yd. at present prices, and including steel armor. Introducing this value of a ($4.00) in Equation 15 gives $5.88 per cu. yd. for the value of x. The cost of concrete in the Duluth superstructure (see Fig. 1, Plate XXII) exceeded this. The cost of concrete in the Superior Entry piers, as far as constructed, has been considerably less than $5.88, and the final results will be noted with interest. Mr. Darling.

J. L. Van Ornum, M. Am. Soc. C. E., St. Louis, Mo.*—Major Gaillard's description of the method of locating and making soundings through ice is very interesting and valuable. Especially pertinent is the reference to the improved ice-boring machine which enables soundings to be taken through ice as fast as is practicable in open water from a boat; the speaker wishes that a fuller description of this improved machine, and of its operation, had been given. It is also to be noticed that this method not only secures the location of every sounding, but allows this to be done more accurately than is possible by the locating of every third or fourth sounding, by angular or stadia location. Mr. Van Ornum.

However, the speaker believes that, even in a latitude where the climate is favorable, the method referred to above would lose its superiority in accuracy if there is a considerable current velocity of the water beneath the ice. From a boat the verticality of the lead-line can be controlled; but this necessity would seem to be practically unattainable in sounding through ice in flowing water. Nor can the speaker believe that sounding from a boat is necessarily more expensive; his experience on several extensive surveys indicates a cost, in water averaging somewhat less than half the depth at Duluth, of about 2 cents per sounding. It is regretted that he has not at hand the record of these soundings, but their number averages between three and four times the number of located soundings, the record of which (by the stadia method) was 980 for 1 day; and, at another time, 200 in 90 minutes; on another survey, the number of soundings averaged more than 40 000 per month for several months. These figures refer to the work of a single sounding party. It is not likely that the greater depth would raise the cost more than from 2 to 3 cents each.

The construction of monolithic piers at the Superior Entry, at probably a less cost per linear foot than for the less enduring rock-filled timber crib with concrete superstructure of the Duluth Canal, is an achievement that is very gratifying. Wherever conditions of

* Prof. of Civ. Eng., Washington University.

r. Van Ornum navigation and of harbor improvements have reached a position where future modification will be confined to narrow limits, the permanent construction should replace the more tentative and temporary which is so admirably adapted to the era of development and adjustment.

Mr. Coleman. CLARENCE COLEMAN, M. AM. SOC. C. E., Duluth, Minn. (By letter.)—In 1894-95-96-97, the writer, having charge of the construction of the concrete superstructure of the United States breakwater at Marquette, Mich., under Major (now Lieutenant-Colonel) Clinton B. Sears, Corps of Engineers, U. S. Army, built in place a considerable amount of subaqueous Portland cement concrete to serve as the foundation for the monoliths of concrete which compose the superstructure of that breakwater. The subaqueous concrete was laid to a depth of 2 ft. over the entire upper surface of the stone-ballasted cribs, the surface of the water being generally from 2¾ to 3 ft. above the top of the reduced cribs. This concrete was deposited from a cubical steel bucket of 1 cu. yd. capacity, designed with a tripping bottom composed of two leaves which opened outward when released, allowing the concrete to slide out with a minimum of disturbance.

The writer has used a tremie at various times for depositing subaqueous concrete, but has not been able to obtain such uniformly good results as with a properly designed bucket. For use at Marquette, in 1894, he designed a telescopic tremie arranged so that the lower joint of the telescoping cylinders could always be filled with concrete before entering the water. This machine was supported on a truck designed to move on a track table, and the table in turn moved on its own track at right angles to the direction of the truck track, which it carried; so the area to be concreted was only limited by the gauge and travel distance of the table.

The concrete was composed of Portland cement, a rather coarse sand and broken stone, in the proportion of 17 lb. of cement, ½ cu. ft. of sand, and 1 cu. ft. of broken stone.

The lake water being very clear at the location of this work, ample opportunity was afforded for very thorough inspection of this concrete, and the result was so satisfactory that the writer has since constantly advocated the use of subaqueous moulded monolithic concrete blocks surmounted with concrete superstructure for piers and breakwaters on harbor works for the Great Lakes. In November, 1901, Captain (now Major) D. D. Gaillard, Corps of Engineers, U. S. Army, being then in charge of the improvement of rivers and harbors on Lake Superior, authorized the writer to construct a subaqueous block of monolithic concrete at the west end of the North Pier of the Duluth Ship Canal.

The location of this monolith was in water 20 ft. deep. The

Mr. Coleman.

season was most inauspicious, and the plant available not such as could have been desired.

A mould was hurriedly constructed of old decking plank from the North Pier. This mould was ballasted with gravel carried in hollow walls and aligned and leveled in place. The concrete was composed of 20 lb. cement, ½ cu. ft. of sand and 1 cu. ft. of pebbles, mixed very wet and deposited with steel buckets of 1 cu. yd. capacity with tripping bottoms, as described for the work at Marquette. When the mould to this block was removed, in April, 1902, the water was very clear, and the concrete could be seen quite distinctly for a depth of 12 or 15 ft., the surfaces presented a good appearance, and all tests that have been made since its construction demonstrate the concrete to be of first-class quality. The success of this block had a determining influence with Major Gaillard in the adoption of the concrete piers at Superior Entry, Wis., a cross-section of which is shown in his paper, Fig. 2, Plate XXII.

The "new all-concrete piers" at Superior Entry, Wis., so well and tersely described, and now under construction, are for the definition and permanent maintenance of the canal which connects Lake Superior with the southerly portion of Duluth-Superior Harbor. They are located at the site of the natural entrance (see Fig. 1, Plate XX). The canal, as now being improved, will be 3 233 ft. long, 300 ft. wide between parallel piers for a distance of 2 248.23 ft. and widening at the harbor end with curves of 1 063 ft. radius for each pier; the length of curve on North Pier is 1 194.62 ft., on South Pier, 774.62 ft. The length of North Pier is 3 442.85 ft.; of South Pier, 3 022.85 ft.

The volume of concrete, as computed from cross-sections for both piers and the accessory works of sea and harbor walls, will slightly exceed 100 000 cu. yd. A depth of 23 ft. will be maintained in the canal.

The South Pier concrete work was commenced on June 1st, and closed on September 21st, 1904, a total length of 1 584 lin. ft. of the South Pier having been completed. This comprises 99 subaqueous blocks, 123 superstructure blocks, 15 554.77 cu. yd. of subaqueous and 3 049.68 cu. yd. of superaqueous concrete, or a total of 18 604.45 cu. yd. in place.

The procedure for this work consisted in dredging a trench 21 ft. below low-water datum for the pier site, jetting and driving the bearing piles, butt end down, to 15 ft. penetration in bottom of trench, cutting off bearing piles 3 ft. above the bottom of trench, as shown typically in Fig. 2, Plate XXII. Trestles were then constructed and tracks laid thereon for carrying the plant for handling concrete moulds, etc.

The plant for performing this work was for the most part of special design, and, in the main, may be described as:

Mr. Coleman. A mould-setting traveler equipped with an engine for hoisting and lowering by means of four drums acting synchronously, or independently, as desired. These drums were actuated from the engine by worm gears, and had a speed in hoisting or lowering of 4 ft. per minute. The same engine furnished the power for propulsion which was applied to eight of the twelve traction wheels through gears and sprocket chains; speed on truck, 90 ft. per minute. The trestles and tracks were so arranged that this traveler spanned the pier trench with its 31-ft. gauge, the transverse adjustment of the suspended moulds was accomplished through suspension trolleys and the longitudinal adjustment through the movement of the traveler on its track. The wheel base was 44 ft. and the maximum curvature of the track, 563 ft. radius. The four forward trucks were swivelled on steel bedplates, and the two rear trucks were provided with idler wheels sliding on axles of sufficient length to allow for maximum curve, a differential gear on the main driving shaft compensating for curvature.

This machine proved very satisfactory in every respect, setting, removing, and assembling the moulds with surprising dispatch. It required a crew of five men for its operations. The time required for setting a subaqueous mould rarely exceeded one hour, and the work of removing and reassembling a mould has been done in 45 minutes (see Fig. 1, Plate XXXIII).

Two steel traveling derricks of 14-ft. gauge and 10-ton capacity, on a fixed boom, 31 ft. from center of track, mounted on Gantry frames, giving a clearance of 6 ft. 6 in. above rails, full circle swing of boom, traction speed, 400 ft. per minute, hoisting speed, 50 ft. per minute. These travelers spanned with their gauge double tracks of 3-ft. gauge and allowed the locomotives with their trains of concrete cars to pass under them without interference with their work. They were designed for handling the buckets of concrete and parts of moulds. The tracks for all the travelers and the concrete tracks were so arranged that at no time could the operations of any part of the plant interfere with the work of any other part (see Figs. 1 and 2, Plate XXXIV).

The moulds for the isolated substructure blocks were bottomless boxes framed in four pieces of heavy timber and lined with 2 by 8-in. plank. The sides were held against the trapezoidal end pieces by $1\frac{1}{4}$-in. steel tie-rods with turnbuckles passing through and acting on beams fastened to the side pieces.

These rods were provided with eyes at each end, in which wedge-bolts were inserted. These wedge-bolts could be withdrawn by a screw-jack, from above the water when it was desirable to remove the moulds, and the turnbuckle tie-rods lifted out by a rope fastened at the center of the rod. This left the moulds free to be picked up

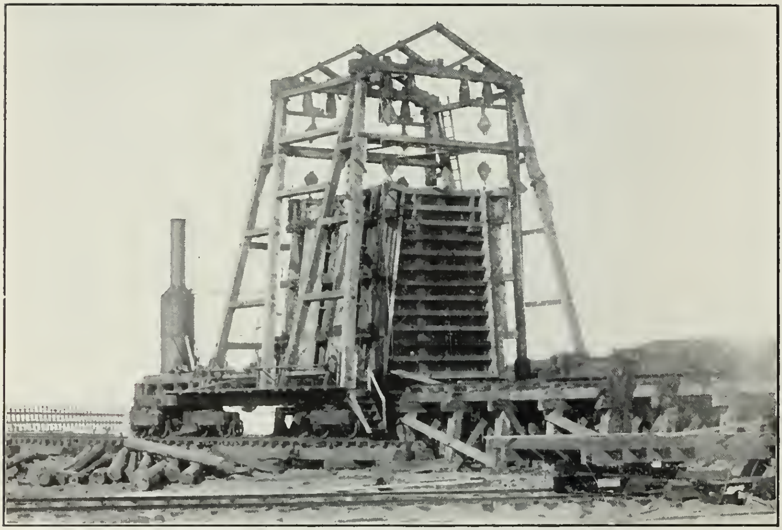


Fig. 1.—Removing and Reassembling Concrete Moulds, Duluth-Superior Harbor.

Fig. 2.—Removal of Subaqueous Moulds, Duluth-Superior Harbor.

and reassembled on the mould-setting traveler. The entire work of removing these subaqueous moulds in 23 ft. of water was performed throughout the season without delay and without having recourse to a diver (see Fig. 1, Plate XXXIII). Mr Coleman.

The moulds for the intermediate subaqueous blocks were two of the side pieces used for the isolated blocks, held apart to the proper batter by 6 boxes of 1-in. plank of 4 by 4-in. inside dimension. The turnbuckle tie-rods passed through these boxes, and when set up, the boxes acting as struts, held the side pieces to conform to the trapezoidal cross-section of the isolated blocks, so the mould could be set in place with its open ends projecting 1 ft. over the ends of two isolated blocks and then filled with concrete (see Fig. 2, Plate XXXIII). When the time came for removing an intermediate mould, the tie-rods were released by a device similar to that described for the isolated moulds, the rods were withdrawn from the boxes and the side pieces picked up by the mould traveler and reassembled for setting. The superstructure moulds were built to conform to the proper cross-section, and the plan of constructing isolated and intermediate blocks was pursued. The moulds for isolated blocks were set up complete on the trestles, and the mould was picked up entire by one of the 10-ton traveling derricks and set in place (see Fig. 2, Plate XXXIV). The intermediate moulds were formed by placing two side pieces of an isolated mould against the ends of two isolated blocks, where they were secured by anchor bolts with threaded ends, nuts, and washers set in the isolated blocks at time of moulding. The work of removing and resetting the superstructure moulds was performed by either of the two steel travelers or the mould traveler, as happened to be most convenient. The concrete buckets were of steel, twelve in number, with a capacity of 70 cu. ft. per bucket. Two leaves hinged to opposite sides formed the bottoms; they could be easily tripped with a cord fastened to a spring latch when used in the subaqueous work, and by simply releasing the latch by hand when used in the superstructure. Concrete trains were made up of a locomotive and two cars carrying two buckets on each car, or about 10 cu. yd. per train. One locomotive and six cars were used for the transportation of concrete from the mixer (see Fig. 1, Plate XXXIV). The concrete tracks passed directly under the mixer and out on a trestle alongside the line of the pier (see Fig. 1, Plate XXXV). The buckets were loaded at the mixer by simply lifting a lever, which allowed a mass of concrete to slide from a steel-lined chute, directly into the bucket.

The concrete mixer was an improved cubical mixer, revolving on trunnions about an axial line through two diagonal corners of the cube. This machine received and discharged the batches of concrete without stopping or variation of speed, and the concrete could

Mr. Coleman. be seen throughout the entire process of mixing. The mixer was charged by measuring chutes from the platform above, as to sand and pebbles, and the cement was weighed in from the same platform by sack units. One man operated the mixer, releasing a lever which admitted the dry aggregates, and pulling a cord which allowed a gauged quantity of water to flow in under head through a 2-in. pipe. The gauging apparatus for the water was so arranged that a definite quantity of water, as percentage by weight of cement, could be introduced and regulated to conform to the varying hygrometric condition of the sand (see Fig. 1, Plate XXXV).

The pebbles were hauled by cable up an inclined trestle in trains of two cars, each of 2 cu. yd. capacity, and dumped into a bin located above the material-assembling platform. The sand was hauled up an incline and stored in the same manner. Two ordinary hoisting engines furnished the power for this haulage and also pumped the water and actuated the cement elevator.

The pebble incline terminated under the stock pile of pebbles, where the cars were loaded by raising doors to chutes fixed in the roof of the gallery. Two men attended the cars in the loading gallery. The sand was shoveled into the cars by three men at the terminus of the sand track. The cement track was through the center of the cement warehouse and led out to the foot of the cement elevator, where the bags were placed on a bag elevator, elevated and dumped on the assembling platform. From two to three men were required for loading the cement on a car in the warehouse and delivering it on the cement elevator. On the assembling platform, two men measured the batches of sand and pebbles by the simple operation of levers which controlled the chutes from the storage bins, and three men took the cement from the cement elevator and delivered it into the mixer hopper. One of these men also took care of the empty bags, which were put up in bales containing 50 bags, for return to the cement contractor. Two men attended the concrete sub-hopper and chute under the mixer for loading cars. There was one engineman on engine of concrete mixer, and two enginemen on pebble and sand haulage. One man attended the concrete buckets as they came in on the storage track, examining latches and sanding the bottom to prevent the adhesion of cement when deposited under water.

The materials composing the concrete were water-worn pebbles, water-worn sand, and Portland cement. The pebbles are from the detritus of the igneous rocks which abound on the north shore of Lake Superior. They are well-shaped, very hard, from ¼ in. to 3 in. in greatest diameter. They are loaded on deck scows with a clam-shell dredge and transported to the work, where they are unloaded with a clam-shell dipper into a hopper feeding into a bucket elevator

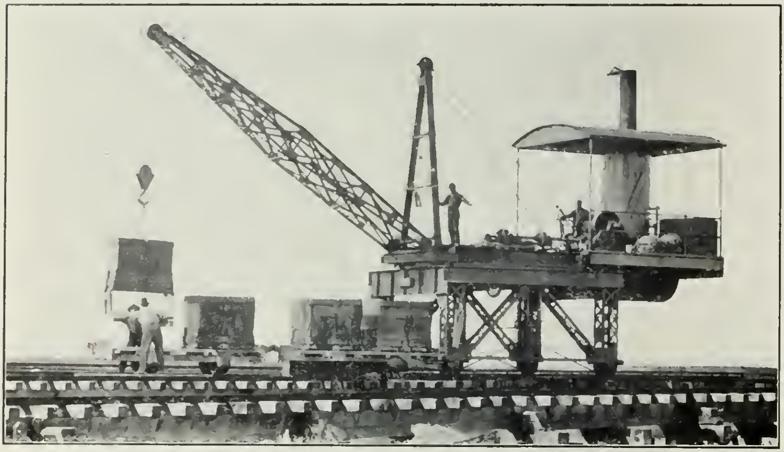


FIG. 1.—TRAVELING DERRICK HANDLING CONCRETE BLOCKS, DULUTH-SUPERIOR HARBOR.

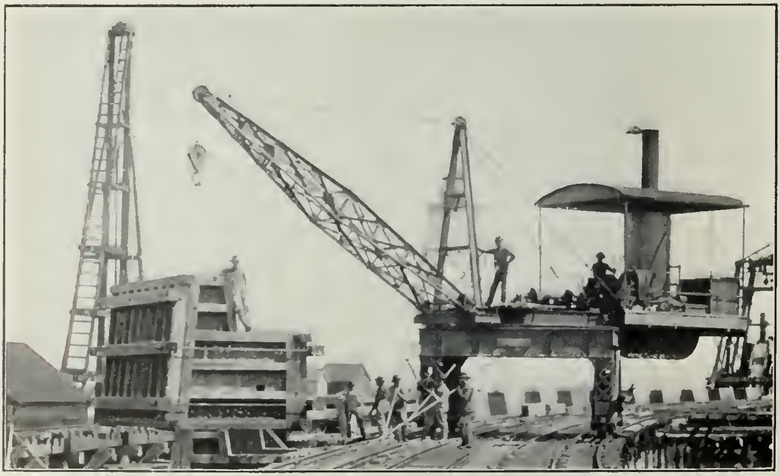


FIG. 2.—MOULDS FOR ISOLATED BLOCKS, DULUTH-SUPERIOR HARBOR.

which lifts them to a rotating cylindrical double screen, where a jet of water from a 4-in. pipe, under pressure from a powerful pump, cleans the pebbles very thoroughly. From the screen they are discharged through chutes into cars of 1 cu. yd. capacity and hauled by cable up an incline track, 65 ft. high, and automatically dumped on the stock pile. The sand used in the concrete was pumped out from the harbor basin and the pier trench excavation and deposited on Wisconsin Point with a view to its use in the pier construction. Mr. Coleman.

The writer has for many years been convinced of the economic advantages of water-worn sand and pebbles in concrete construction, as compared with crushed sand and stone, always provided that the geological origin of the stone, from which these aggregates are derived, is as good in one case as the other. The writer made a demonstration* intended to show the relative economic values, for the purposes of concrete, of water-worn pebbles and sand as compared with stone and sand crushed artificially and considered from their volumetric, gravimetric and granulometric conditions, with reference particularly to the quantity of cement required. Using the piers of the Duluth Ship Canal as an illustration in the demonstration referred to, an economic advantage of $30 058 in saving of cement alone was shown by the use of pebbles and water-worn sand.

The writer suggests the following reasons as pertinent to the economic advantages of pebbles, as compared with crushed stone:

Prime cost where pebbles are available; reduction of voids, and consequent reduction of mortar; greater strength of the particles due to their granulometric condition; absence of planes of weakness common to crushed materials, greater facility with which they may be cleansed; as allowing mortar to more thoroughly surround the particles and fill the voids in the process of compacting.

In very extended tests made by the writer at the U. S. Cement Testing Laboratory at Duluth with concrete briquettes from which pebbles larger than ¾ in. had been removed, mortar composed of 1 part cement to 3.43 of sand by weight, it was shown that generally after 28 days, pebbles embedded in the breaking section of the briquettes were broken in the tests for tensile strength.

The pebbles used in these tests were very hard, smooth, and strong, and, from these results, it was concluded that for concrete in which stone is the principal element, when the adhesion of the matrix or binding media on the surfaces of the stones is as strong as the stone itself, then the quality of the concrete, as to its ability to resist stresses in tension or compression, cannot be increased without the introduction of new elements.

The proportions of the dry aggregates which compose the concrete for the piers at Superior Entry are as follows:

* Annual Report, Chief of Engrs., U. S. A., 1899, Pt. 3, pp. 2645–2647

Mr. Coleman.

For the Subaqueous Concrete:

By Weight:

Cement to sand, 1 to 2.845;
Cement to pebbles, 1 to 5.775;
Cement to sand and pebbles, 1 to 8.620.

By Volume:

Cement to sand, 1 to 2.50;
Cement to pebbles, 1 to 5.00;
Cement to sand and pebbles, 1 to 7.50.

And for the superaqueous concrete:

By Weight:

Cement to sand, 1 to 3.555;
Cement to pebbles, 1 to 7.218;
Cement to sand and pebbles, 1 to 10.773.

By Volume:

Cement to sand, 1 to 3.12;
Cement to pebbles, 1 to 6.25;
Cement to sand and pebbles, 1 to 9.37.

The weight per cubic foot of cement assumed as 100 lb.

The weight per cubic foot of sand determined as 113.69 lb.

The weight per cubic foot of pebbles determined as 115.50 lb.

The specific gravity of the subaqueous concrete was determined as 2.3736, and the weight per cubic foot, 148.35 lb.

The specific gravity of the superaqueous concrete was determined as 2.416, and the weight per cubic foot, 151 lb.

The weight per cubic foot and the determination of voids for sand and pebbles were made up from three averages of specific gravity, as determined with the Le Chatelier apparatus for the said matter composing them. And three averages of specific gravity, as determined for the loose materials considered as a volume, or $\frac{w}{v\,m} = g$

g = specific gravity of volume of loose particles.
g' = specific gravity of solids composing volume.
m = weight of 1 cu. ft. of water.
w = weight of loose particles in any given volume.
w' = weight of a solid of equal volume.
n = percentage of voids in the volume of loose particles.
v = volume.

Then $$\frac{w}{v\,m} = g, \text{ and } g \times m = \frac{w}{v}$$

$$g' \times m = \frac{w'}{v}$$

$$n = \frac{w' - w}{w'}$$

Mr Coleman.

TABLE 22.—MORTAR TESTS.

Number of test.	Volume of sand, in cubic inches.	Weight of sand, in grammes.	Percentage of voids in sand.	Fineness of Sand. Percentage passing No. 10 Sieve.	Fineness of Sand. Percentage passing No. 20 Sieve.	Fineness of Sand. Percentage passing No. 30 Sieve.	Weight of Sand, per Cubic Foot. As determined by weighing.	Weight of Sand, per Cubic Foot. As determined by specific gravity.	Specific gravity of sand.	Parts by weight of sand to one of cement.	Fineness of Cement. Percentage passing No. 50 Sieve.	Fineness of Cement. Percentage passing No. 100 Sieve.	Fineness of Cement. Percentage passing No. 200 Sieve.	Specific gravity of cement.	Pounds of cement to ½ cu. ft. of sand.	Weight of cement, in grammes.	Percentage by weight of cement to sand.	Percentage of water to cement.	Percentage of water to sand.	Percentage of water to sand and cement.	Weight of water to sand and cement, in grammes.	Volume of mortar, in cubic inches.	Percentage of increase in volume of mortar.
t	*a*	*b*	*c*	*d*	*e*	*f*	*g*	*h*	*i*	*j*	*k*	*l*	*m*	*n*	*o*	*p*	*q*	*r*	*s*	*t*	*u*	*v*	*w*
1.....	216	6 361	32.388	100	69.66	43.62	112.188	112.868	2.669	1	99.94	93.60	77.48	3.106	14.00	6 361	100.00	12.50	5.66	9.07	1 155	357.00	65.28
2.....	216	6 361	32.388	100	69.66	43.62	112.188	112.868	2.669	1½	99.94	93.60	77.48	3.106	9.33	4 240	66.65	12.50	5.66	8.39	890	295.48	36.79
3.....	216	6 361	32.388	100	69.66	43.62	112.188	112.868	2.669	2	99.94	93.60	77.48	3.106	7.00	3 180	49.99	12.50	5.66	7.94	758	262.98	21.75
4.....	216	6 361	32.388	100	69.66	43.62	112.188	112.868	2.669	2½	99.94	93.60	77.48	3.106	5.60	2 544	39.99	12.50	5.66	7.61	678	245.57	13.69
5.....	216	6 361	32.388	100	69.66	43.62	112.188	112.868	2.639	3	99.94	93.60	77.48	3.106	4.66	2 120	33.32	12.50	5.66	7.36	625	235.95	9.23
6.....	216	6 361	32.388	100	69.66	43.62	112.188	112.868	2.669	3½	99.94	93.60	77.48	3.106	4.00	1 817	28.57	12.50	5.66	7.17	587	228.50	5.79
7.....	216	6 361	32.388	100	69.66	43.62	112.188	112.868	2.669	4	99.94	93.60	77.48	3.106	3.50	1 590	24.99	12.50	5.66	7.04	559	224.79	4.07
8.....	216	6 361	32.388	100	69.66	43.62	112.188	112.868	2.669	4½	99.94	93.60	77.48	3.106	3.33	1 414	22.22	12.50	5.66	6.90	537	221.70	2.64
9.....	216	6 361	32.388	100	69.66	43.62	112.188	112.868	2.669	5	99.94	93.60	77.48	3.106	2.80	1 272	19.99	12.50	5.66	6.79	519	221.01	2.32
10.....	216	6 361	32.388	100	69.66	43.62	112.188	112.868	2.669	5½	99.94	93.60	77.48	3.106	2.54	1 157	18.18	12.50	5.66	6.71	505	220.92	2.28
11.....	216	6 361	32.388	100	69.66	43.62	112.188	112.868	2.669	6	99.94	93.60	77.48	3.106	2.33	1 060	16.66	12.50	5.66	6.64	493	220.45	2.06
12.....	216	6 361	32.388	100	69.66	43.62	112.188	112.868	2.669	6½	99.94	93.60	77.48	3.106	2.13	978	15.37	12.50	5.66	6.56	482	220.42	2.04
13.....	216	6 361	32.388	100	69.66	43.62	112.188	112.868	2.669	7	99.94	93.60	77.48	3.106	2.00	908	14.29	12.50	5.66	6.52	474	219.31	1.53
14.....	216	6 361	32.388	100	69.66	43.62	112.188	112.868	2.669	7½	99.94	93.60	77.48	3.106	1.86	848	13.33	12.50	5.66	6.46	466	219.57	1.65
15.....	216	6 361	32.388	100	69.66	43.62	112.188	112.868	2.669	8	99.94	93.60	77.48	3.106	1.75	795	12.49	12.50	5.66	6.41	459	219.21	1.49

Mr. Coleman.

TABLE 22.—MORTAR TESTS (*Continued*).

Number of test.	Increase of volume, in cubic inches.	Residue of mortar after moulding 1/8-cu. ft. cube, in grammes.	Weight of cube, in grammes, 3 months in air at 80° fahr.	Increase of weight of cube, in grammes, 15 days in water at 65° fahr.	Percentage by weight of water remaining after 3 months in air.	Percentage of water evaporated after 3 months in air at 80° fahr.	Weight of cube, in grammes, after 15 days in water at 65° fahr.	Percentage of absorption after 15 days in water.	Compressive Strength of 6-in. Cube. Age when broken.	Compressive Strength of 6-in. Cube. Ultimate strength, in pounds.	Tensile Strength per Square Inch. 7 days.	28 days.	3 months.	6 months.	1 year.	2 years.	Rate of stress per minute.	Number of briquettes averaged.	Coefficients, tensile to compressive strength.	Remarks. Cubes.
A	x	y	z	a'	b'	c'	d'	e'	f'	g'	h'	i'	j'	k'	l'	m'	n'	o'	p'	
1.....	141.00	5 481	8 268	103	7.41	1.66	8 371	1.24	2 years and 5 months.	409 200	568	613	744	767	784	799	400	5	14.2	1A to 7A, inclusive, tested at Watertown Arsenal, Mass., May 24th, 1902; compressed surfaces faced with plaster of paris.
2.....	79.48	3 091	8 254	129	6.50	1.89	8 383	1.56	" "	374 500	459	557	692	728	756	711	400	5	14.6	
3.....	46.98	1 840	8 281	156	5.53	2.41	8 437	1.88	" "	342 000	407	507	609	678	629	627	400	5	15.1	
4.....	29.57	1 154	8 229	174	5.05	2.56	8 403	2.11	" "	292 100	279	378	430	525	536	518	400	5	15.6	
5.....	19.95	770	8 106	225	4.14	2.92	8 331	2.77	" "	221 870	239	302	102	429	411	409	400	5	15.07	
6.....	12.50	480	8 020	273	3.74	3.43	8 293	3.40	" "	226 000	183	257	331	321	317	328	400	5	19.1	
7.....	8.79	333	7 874	326	2.97	4.06	8 200	4.14	" "	188 500	139	211	269	299	255	272	400	5	19.2	
8.....	5.70	214	7 786	362	2.78	4.12	8 148	4.65	2 years.	151 000	119	166	199	236	223	207	430	5	20.2	
9.....	5.01	185	7 637	426	2.37	4.42	8 063	5.58	"	129 000	101	139	184	208	166	191	400	5	18.7	
10.....	4.92	179	7 471	466	1.64	5.08	7 937	6.23	"	90 640	87	130	151	149	153	156	400	5	16.1	8A to 15A, inclusive, tested at the Michigan College of Mines, Houghton, Mich., Dec. 24th, 1901.
11.....	4.45	160	7 412	481	1.94	4.70	7 893	6.49	"	100 600	79	110	121	119	119	136	400	5	20.5	
12.....	4.12	157	7 306	511	1.58	4.98	7 817	6.99	"	87 400	62	83	101	103	105	106	400	5	22.8	
13.....	3.31	117	7 269	538	1.53	4.72	7 807	7.40	"	78 000	52	88	98	93	81	122	400	5	17.7	
14.....	3.57	125	7 199	541	1.51	1.95	7 740	7.51	"	69 000	58	76	84	75	92	103	400	5	18.6	
15.....	3.21	112	7 151	563	1.42	4.99	7 714	7.87	"	62 000	43	66	78	74	80	105	400	5	16.4	

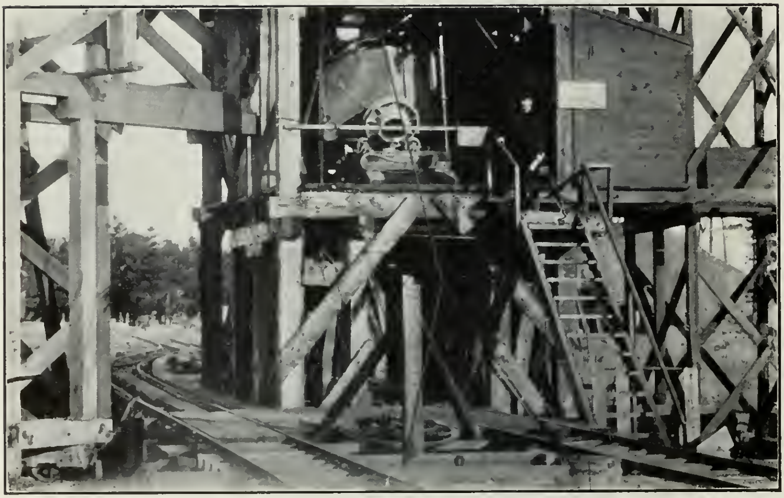


FIG. 1.—CONCRETE MIXER, DULUTH-SUPERIOR HARBOR.

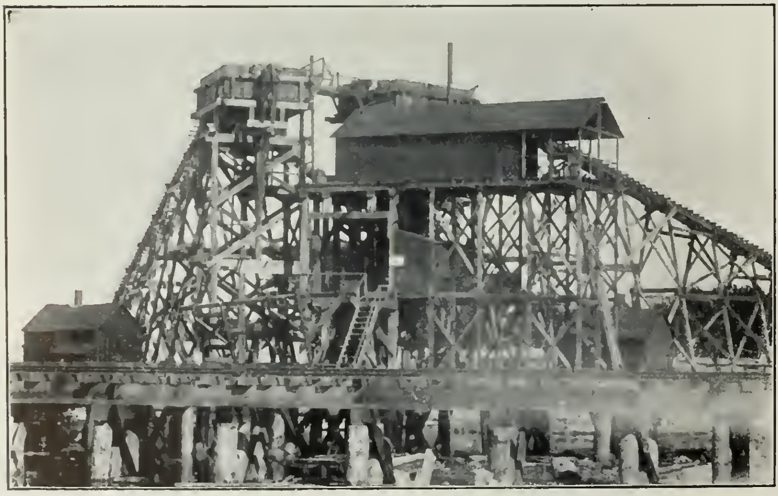


FIG. 2.—STAGING FOR CONCRETE MIXER, INCLINE TRESTLE FOR SAND AND PEBBLES, AND CEMENT ELEVATOR, DULUTH-SUPERIOR HARBOR.

Mr. Coleman.

Table 22 was made by the writer from the results of a very careful set of tests to determine the characteristics of Portland cement mortars. The greatest difficulty encountered in conducting these tests was to establish a constant for water, as percentage of sand and cement, respectively. The constants, *r* and *s*, as shown in Table 22, worked out very perfectly, and the same degree of plasticity was apparent in Test No. 15 as in Test No. 1.

Considering volumetrically and gravimetrically a cubic foot of each of the two classes of concrete used at Superior Entry:

For the Subaqueous Concrete:

Cement, weighing	100.00 lb. per cu. ft.	.....	20.00 lb.
Sand, weighing	113.69 " " " "	.....	56.84 "
Pebbles, weighing	115.50 " " " "	.....	115.50 "
Total weight		.....	192.34 lb.

The actual voids in the pebbles were 32 per cent. The volume of sand was 50% of the pebbles. This gives an excess of sand amounting to 18%, and 18% of 192.34 = 34.62. The increase for the volume of mortar as shown under *w*, Table 22, lies between 13.69 and 9.23%, and is taken as 11.46, and 11.46% of 76.84, the combined weight of cement and sand, = 8.81, and 192.34 — (34.62 + 8.81) = 148.91 lb. for 1 cu. ft. of concrete. This agrees within 0.56 lb. of the weight per cu. ft., as computed from the specific gravity of the concrete.

For the Superaqueous Concrete:

Cement, weighing	100.00 lb. per cu. ft.	.....	16.00 lb.
Sand, weighing	113.69 " " " "	.....	56.84 "
Pebbles, weighing	115.50 " " " "	.....	115.50 "
Total weight		.....	188.34 lb.

Actual voids in pebbles = 32%, volume of sand 50% of volume of pebbles. Excess of sand, 18%, and 18% of 188.34 = 33.90. The increase for the volume of mortar, as shown under *W*, Table 22, is 5.79%, and 5.79% of 72.84, the combined weight of cement and sand, is 4.22 lb., and 188.34 — (33.90 + 4.22) = 150.22 lb. for 1 cu. ft. of the superaqueous concrete. Which agrees within 0.88 lb. with the weight per cubic foot for concrete as computed from the specific gravity.

From June 1st to September 21st, 1904, a total of 18 083.81 cu. yd. of concrete was built in place. The actual running time of the concrete mixer was 34 138 minutes; this gives a rate of 1 cu. yd. of con-

Mr. Coleman. crete for every 1.88 minutes of time, when the machine was in operation. Of the total amount of concrete built, 83.6% is below the surface of the water. The total quantity of cement used was 23 295.75 bbl. of 375 lb. each. This, divided by the total cubic yards of concrete in place, $\frac{23\,295.75}{18\,604.45} = 1.252$ bbl. for each cubic yard in place.

Plate XXXVI shows the plan of building the isolated monoliths of the superstructure; the blocks are covered with burlap, which is kept thoroughly wet for ten days. Fig. 2, Plate XXXV, shows the concrete-mixer staging, the sand and pebble incline trestles leading to the bins at top of staging and the cement elevator.

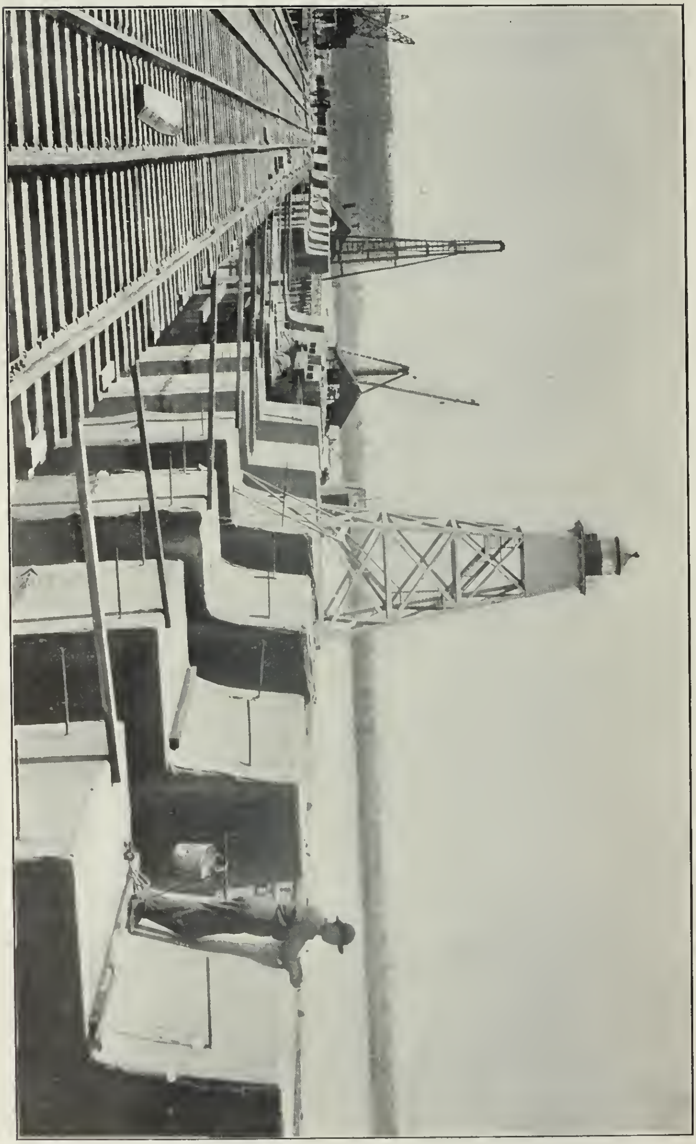
The concrete-pier construction at Superior Entry has been in the writer's charge under the direction of Major Charles L. Potter, Corps of Engineers, U. S. Army, in charge of river and harbor improvements on Lake Superior.

The writer concurs with Major Gaillard as to the probable cause of the greatly increased strength of ice briquettes, when finely divided particles are introduced into the water to be frozen.

For the purpose of further investigation, the writer made some tests by filling the briquette moulds with 2 g. of absorbent cotton, and then filling them with water and exposing them to temperature of —15° fahr. for 90 minutes, and then breaking them on the testing machine. Three briquettes showed, respectively, 500, 477 and 337 lb. tensile strength per sq. in. It is evident that the more finely divided the particles that may be introduced into these tests of tensile strength for ice, the greater will be the interference with the normal arrangement of the ice crystals, and consequently the greater the increase of strength.

Mr. Carey. A. E. Carey, M. Inst. C. E., London, England. (By letter.)—The opening portion of Mr. Vedel's paper on "Island Harbours" follows closely upon the lines of the recent discussion at the Institution of Civil Engineers on "The Sanding-Up of Tidal Harbours."

With regard to the island harbours described by Mr. Vedel, and the contours of accumulation in relation to such work at which he arrives on theoretical grounds, granting the premises of Mr. Vedel's calculations, his conclusions seem unanswerable. It must not be forgotten, however, that the three harbours described are in somewhat exceptional localities, as they are projected into the waters of the Kattegat and the Baltic Sea, respectively, which are to a large extent sheltered and landlocked. It must be further borne in mind that these are essentially fishery harbours, the enclosed basins having only depths varying from 4½ ft. to 8½ ft.; their design, therefore, hardly provides a precedent for the construction of similar works on an extended scale, having for their object the reception of modern vessels of any considerable draft.



METHOD OF PLACING ISOLATED MONOLITHS OF SUPERSTRUCTURE, DULUTH-SUPERIOR HARBOR.

Mr. Vedel does not give the height of the walls enclosing these island harbours, but it is obvious that unless the enclosing walls were of very considerable height above the water-line they would fail to provide the wind shelter which is essential for the safe handling of large ships. The entrances to such harbours would also have to be greatly contracted, as is the case in the instances cited, otherwise, if built in localities of ocean swell, they would be too rough to permit of large vessels riding in safety. On this point, it is curious to note that at Madras Harbour the authorities were actually compelled to send ships to sea for safety, owing to its exposure during monsoons. Mr. Carey

A small fishing-boat can lie in a harbour of the description indicated in Mr. Vedel's paper comparatively snugly, as its hull would be protected from the wind by the walls of the moles, but an ocean going steamer of any considerable draft would be necessarily much exposed.

The lesson of the repeated failures in harbour construction in all parts of the world seems to be that the dominant factor is after all the natural condition of a given locality, and that an attempt to build a purely artificial structure, in a spot unsuited by natural conditions for such a work, is only possible under great limitations. A project to fence off a piece of the open sea by solid works, and throw a viaduct from such enclosure to the shore, is necessarily limited in utility and in possibility.

The alternative suggestion which the writer ventured to make before the Institution of Civil Engineers is that in many such localities reasonable trading facilities can be created by a skeleton structure, such as that now existing at Port Elizabeth, South Africa. Such a structure is of necessity somewhat limited in its utility, but a little experience would enable navigators to operate traffic with reasonable certainty and safety. It is not necessary to state that the design of moorings and cranage and the general design of the structure would have to be special.

E. L. CORTHELL, M. AM. SOC. C. E., New York City. (By letter.) The writer differs from Major Gillette in respect to several statements made in his paper. Mr. Corthell

Referring to page 300, a "littoral current" is not caused by winds, at least, not generally, and it is doubtful whether it is ever caused by them. A true littoral current is either an ocean current passing along the shore in its general progress, or it is a reflex current from a distant ocean current. Such a current may move directly against the wind, instead of with it.

The Gulf current, a true littoral current, passes into the Gulf of Mexico, around the Promontory of Yucatan, passes along the shores of the Bay of Campeachy and port of Vera Cruz, in a steady, northerly movement. The writer has personally known this north-

Mr. Corthell. erly movement to be strong, continued and persistent at a velocity of probably 3 miles per hour after a "norther" has been blowing straight against it for several days. This current leaves the shore about opposite Tampico and swings around in a circle about 100 miles offshore at Galveston and makes its way toward the outlet at the Florida Straits. Opposite the mouth of the Mississippi River it is well offshore, but there is a nearly constant reflex current, or Gulf current eddy, westward. The writer has found this current strong enough, even in times of calm, to render it difficult to make soundings in 50-ft. depths off the South Pass, the current flowing westward at the rate of at least 2½ miles per hour. The writer was engaged during four years upon the works at the mouth of the South Pass and in not one instance were wreckage and floating objects found to the eastward; always to the westward and, at times, as far west as Brazos de Santiago, several hundred miles distant. The testimony of the officers of the light-house tenders was to the same effect. Their buoys, which got adrift anywhere off this coast and as far east as Mississippi Sound, would always be found to the westward. Major Gillette, on page 323, again refers to this subject and offers the suggestion (apparently for the first time) that the general southeasterly trend of the river in its lower 50 miles may be caused by the action of this westerly "littoral" current. This is not a new suggestion, for Mr. Eads stated it and proved it clearly over thirty years ago, and showed then that this was surely the effect of the western littoral current's depositing the larger part of the sediment discharged from the mouths of the Passes to the westward and thus forming a shoulder, which pushed the river eastward. The writer contends, as did Messrs. Eads and Bayley, that this "littoral," or reflex eddy, current, was one of the causes of success at the South Pass, and, therefore, he must dissent from the following conclusions of Major Gillette (see page 301):

"It follows that such other littoral currents, real or imaginary, should have very little influence upon rational theories of bar improvement."

Upon this subject of "twin jetties," page 305, the difficulty arising from the deposit of material outside and the formation of a new bar, if the sea slope be slight, is referred to. Generally, this need not be apprehended. The littoral or wind currents will generally carry this deposit away from the channel entrance, as they have done for 25 years at the mouth of the Mississippi River and decidedly so at Tampico.

On page 321, it is stated that:

"There is no method yet devised, except possibly dredging, that will not 'push the bar seaward,' and twin jetties, being the most suc-

cessful structural agency yet invented for removing the bar, are doubtless the worst sinners in this respect." Mr. Corthell.

The writer must take exception to this conclusion. The real cause of the trouble noted is the erroneous location of the works. The jetties are generally too far apart. This mistake is very common. There is not in that case sufficient increase in velocity to carry the sediment into deep water and the sea currents.

On page 323, a statement is made about the jetties at the mouth of the South Pass of the Mississippi River, which is entirely erroneous. Major Gillette's words are: "Twin jetties, aided very materially by dredging, have constituted the plan of its improvement." Dredging was never contemplated, although the United States Board of Engineers afterward decided that it was admissible under the contract as an auxiliary means of making the channel. The writer quotes the following from his "History of the Mississippi Jetties," page 214:

"Some of the channel results have been obtained by dredging, but the amount of material moved by this means has been so small, being only about 1% of the whole, that we have not included it in the exhibition of results. This artificial appliance has simply hastened a result which the current had ample power to accomplish, as is evident from the fact that it is now able not only to maintain the channel secured, but to still further enlarge it."

The current, the velocity of which was increased by the jetties, moved in all, during the four years of the development of the channel, 7 607 151 cu. yd. of material from the bottom and carried it seaward. There would not have been any dredging done in the jetty channel had not the peculiar conditions existing at the time (not physical by any means) made it financially necessary to hasten the development of the channel by mechanical means.

In respect to the maintenance of the jetty channel, the peculiar terms of the contract with the United States Government made it necessary to maintain a dredge-boat. The peculiarity was that the contract required the constant maintenance of a channel of three dimensions, *viz.*, it was to be made and maintained, 26 ft. deep and 200 ft. wide, at that depth, with a central depth of 30 ft. without regard to width. If at any time the Government Inspector ascertained by soundings that there was a deficit in any one of these dimensions, it was so reported and the daily amount deducted from the payments. This amounted at one time to over $400 per day. If the channel should be 199 ft. wide at the 26-ft. depth, with a central depth of 40 ft., the channel would still be deficient; or, if the 26-ft. channel would be 250 ft. wide and the central depth, 29½ ft., it would still be a deficient channel and the inspecting officer was obliged to report it so. Except for these peculiar conditions, there

Mr. Corthell. would probably have been very little occasion at any time, for navigation purposes, to do any dredging in the jetty channel. The writer is not now referring to the twelve miles through the South Pass, or to the channel at the head of the Pass, where it was required to maintain a navigable channel of 26 ft. The writer refers to the channel through the jetties at the mouth of the South Pass, which have been in every way successful and have done all that was predicted of them by the projector of the works.

As to the advance of the bar, which had been predicted by some engineers, one of whom stated that the bar would advance into the Gulf at a rate of 600 ft. per annum, or 3 000 ft. in 5 years, the facts are, on the contrary, these: The present sea ends of the jetties, and especially that of the east or windward jetty, are over 200 ft. landward of the point where the writer laid the foundation for them 29 years ago. Another important point that might be properly stated in reference to the works at the South Pass is the following: Mr. Eads, in his application to Congress for the privilege of executing the works at the mouth of the river, proposed to make the channel at the mouth of the South-West Pass, and not at the South Pass, the former being about four times the size of the latter, but as the method was considered experimental, he was given the small Pass, greatly to his disappointment, but he predicted at the time, in 1874, that within twenty years from the time he had opened the mouth of the South Pass, the Government would find itself obliged, on account of the increase in the size of vessels that would visit the Port of New Orleans, to open the mouth of the South-West Pass, which it is now doing at a cost of several million dollars.

Mr. Matthews. W. Matthews, M. Inst. C. E., London, England. (By letter.)—The writer agrees with Mr. Meik as to the limited number of British engineers who advocate the view that the movement of beaches is due to the action of tidal and other currents. Nevertheless, this opinion is held, and he believes erroneously so, by certain engineers who have considered the question, and who have read papers thereon before scientific bodies.

Mr. Vedel. P. Vedel, Esq., Aarhus, Denmark. (By letter.)—Replying to Mr. Carey's remarks, the top of the parapet of the moles enclosing the Danish island harbors described is only about 2.5 m. above mean-water level. The hull of a fishing-boat, but not that of a large ship, is thus sheltered from the wind. Such shelter is not, however, always provided in a land-connected harbor. More serious than this question of wind shelter is that of shelter from the sea, which, on account of the deep water and perhaps vertical walls, is particularly rough at a detached work. At Snogebæk (Fig. 2), during a winter storm, the spray from the waves has covered the masts and rigging

Mr. Vedel. of the fishing-boats, berthed at the east mole, with a coat of ice so heavy that the boats were sunk in the harbor. To prevent such a calamity in the future, a breakwater, shown in Fig. 2, was built east of the harbor, and the boats made it a rule, on the approach of bad weather in winter, to take down their masts.

As to Mr. Hunter's remarks, the flood tide undoubtedly transports material, when such tide runs swiftly; but in localities where tides are insignificant, as along the Danish coasts, the drift, which may nevertheless be considerable, is practically due to the waves only.

Granted, that the quantity of mobile material is inexhaustible—which, however, is not necessarily so everywhere—it remains to be proved that dredging is necessary, if a harbor can be so constructed as to leave no obstruction to the free passage of the material after certain deposits and accumulations have formed.

The tidal wave and currents along the shore may form deposits of material in front and to leeward of a detached work, somewhat as if it were a pier of a bridge. Hence a rounded form of the work, such as that at the harbor at Hundested (Fig. 3), is advisable. To make it oblong parallel to the shore would be objectionable, because the current might not always have that direction, and the accumulations due to wave action increase with the length of the work parallel to the shore. A circular or many-sided regular polygonal form is preferable. But for ships entering the harbor, it may be necessary to provide sea room in the direction of their entrance, and for that reason the form may be made somewhat oblong.

Maj. Gillette. CASSIUS E. GILLETTE, ESQ., San Francisco, Cal.* (By letter.)—Mr. Hunter, in his discussion, is inclined to attribute to tidal currents more influence upon littoral drift than is credited to them in the writer's paper. This is probably due to the fact that the paper is limited to harbors in the United States where the tidal currents are not as strong as they undoubtedly are at various points in the vicinity of the British Isles.

The case in which Mr. Hunter expresses much interest, where the tidal rise was only 14 in., with practically no fresh-water flow, is at Aransas Pass, Tex. The writer will discuss it fully in connection with his reply to Professor Haupt.

As to the total quantity of littoral drift, the same remark applies as does to the effect of tidal flow. Its quantity in harbors of the United States has probably been sometimes underestimated, but, even in such aggravated cases as Ceara, Brazil, and Greytown, Nicaragua, the writer believes that the proper way to deal with it is to stop it by a windward jetty built above high-water mark and extending as nearly square across the path of the drift as the conditions of the bar at the time of the improvement will justify. The

* Major, Corps of Engrs., U. S. A.

Maj. Gillette. triangular area filled in will rapidly increase in size as the jetty is extended, and the wind will soon build high sand dunes, so that the impounding capacity becomes exceedingly large. The work need never be done in deep water if the jetty is extended at the proper time—when the drift begins to go around the end of it in dangerous quantities.

Mr. Corthell objects to the idea of a littoral current being caused by wind, and wants to limit the expression to what the writer in his paper spoke of as "great ocean currents or eddies from them." As the word "littoral" means "alongshore," the writer sees no reason to limit its application in any such way. Since an alongshore current must always be created by a diagonal wind, and such a current, being the result of the integration of the breaking waves which stir up the material, must be in a much better position to transport that material than great ocean currents, or eddies, which affect the water farther offshore, and which may act as often against the wind-wave current as with it. Incidentally, it may be remarked that it is quite possible that the trade winds have a great deal to do with the creation of the Gulf Stream itself, which Mr. Corthell denominates as a "true littoral current."

The idea of an ever-ready littoral current off the mouth of a twin-jetty harbor, to take up and carry away all the detritus discharged, is a fascinating one, and has been urged frequently where the existence of such a current was probably wholly imaginary. Mr. Corthell establishes one off the mouth of the Mississippi River with a velocity sometimes as great as $2\frac{1}{2}$ miles per hour. If this current is constant, which it probably is not, it could be expected to carry away the detritus brought into it by a current of equal or less velocity; but, as the current through the South Pass is often as great as $3\frac{1}{2}$ miles per hour, there must be a great deal of the coarser detritus, brought by this swift current, which could never be moved by even a $2\frac{1}{2}$-mile littoral current. As the river carries a great deal of detritus, it is simply impossible that the jetties in this case can prevent the growth of the bar seaward, and this reasoning is fully borne out by the simple fact that, since the jetties were constructed, the fan-shaped area covering about $1\frac{1}{4}$ sq. miles at the mouth of the Pass shoaled, in the twenty years following the construction of the jetties, an average of $17\frac{1}{2}$ ft. over this entire area. The jetties have not yet needed extension, for the following reasons:

1.—There was deep water just seaward of their ends;

2.—The detritus brought down by the river is mostly in suspension, and part of it drifts away;

3.—The coast line of the delta is very irregular, and the exposure to onshore winds is slight—hence the littoral sand drift due to wind waves is not large;

4.—The unusually strong eddy current described by Mr. Corthell would carry away some of the sediment. Maj. Gillette.

These are very unusual conditions, making this point an unusually favorable one for jetty construction; but, nevertheless, the principles stated in the writer's paper apply here as elsewhere, although this particular point was so exceptional to the generality of harbors in the United States that the writer gave it a special description, because, as stated, it "has a regimen peculiar to itself, not covered in some particulars by the above analysis."

As to the question of dredging, the writer does not consider that a fair description of this improvement could omit the material assistance which has been derived from dredging, irrespective of the reasons for its use. A channel 200 ft. wide is certainly required by shipping, as well as by the contract, and if the plan requires dredging, to secure or maintain such a channel, dredging certainly gives "material assistance" in the plan. Between July 8th, 1879, and January 28th, 1901, the dredge worked, to maintain the channel, 7% of the entire time—day and night—moving a total quantity estimated at 3 264 000 cu. yd. This, compared with the total of 7 600 000 cu. yd., stated by Mr. Corthell to have been moved during the formation of the channel, appears to the writer to have been a material assistance.

Mr. Corthell hints at plagiarism in the writer's statement that the mud of the Mississippi settles generally to the westward. The writer was not attempting to give the genealogy of ideas, or a history of observed facts. All that the paper pretends to be is a statement of reliable facts and sound ideas on the subject of ocean bar improvement, as the writer sees them. He was not aware that Mr. Eads, or anyone else, had ever formulated the idea above mentioned, or the suggested proof of it, and Mr. Corthell does not state where, in Mr. Eads' writings, it may be found. However, the only question at stake is whether or not the idea is a sound one, because, if so, it is a matter of some importance.

Mr. Meik and Mr. Hunter find difficulty in discussing the writer's paper on account of its general nature. This was necessary because of the size of the subject and the desirable length of the paper. The writer is somewhat surprised that several of the propositions laid down, which, as far as the writer knows, have not been advanced before, did not call up some opposition. The two in italics on page 304 are far-reaching and important. Their truth or falsity is a matter of moment, and the writer would like to have had the views of others upon them, especially the second one. The writer believes his theory of the predominance of ebb scour to be new: the views of others would be useful. Further light, too, would have been valuable upon the completeness and accuracy of the writer's schedule of signs to determine the direction of the sand drift.

Maj. Gillette. Professor Haupt's discussion opens upon a minor point, *viz.*, the writer's alleged "change of front" on the subject of sand drift. Whether the writer has or has not changed his mind is an unimportant matter, but Professor Haupt's discussion of it is worthy of a moment's consideration, as bearing on the remainder of his discussion and showing the ingenious uses to which judiciously selected quotations from a writer's work can sometimes be put. He quotes from a previous production, a report upon the improvement of Brunswick Bar, Georgia, and his quotation indicates that the writer therein recommended a single lee jetty for bar improvement, whereas he now favors twin jetties constructed so that the windward member should constitute an effective sand stop, the change constituting a "frank change of front."

The writer has made no such change, and, to show that the quotation as given is wholly misleading, it is only necessary to quote a sentence preceding it and another following it on almost the same page of the same report. The writer was discussing every method of improvement that had ever been tried in the United States, giving in each case the theory on which its advocates based their support. Among the plans thus considered was that of a single lee jetty. The preceding sentence was as follows: "The theory upon which ordinary single lee jetties, applicable to Brunswick Bar, have been proposed is about as follows," etc. The sentence which follows the above quotation was, "For these reasons the improvement of this Bar by a single jetty is not recommended."

This is not the first time that Professor Haupt has made this same error. In a paper* discussing the above report he stated as follows: "Since it is again seriously recommended to repeat the mistake by locations made on the leeward side of the channel," etc. And again—"Here, where a jetty was built to leeward according to the author's ideas, the natural forces changed it to windward by shifting the channel to the opposite side, a complete demonstration in his own district." And again, further on, "And still the same method is urged as being the proper policy to pursue." Even a casual reading of the report should have shown that the writer did not make any such recommendations or urge any such methods as are attributed to him by Professor Haupt.

These things indicate that detached quotations should often be considered in connection with the context.

Professor Haupt's discussion of the writer's paper is somewhat disconnected and inconvenient to consider seriatim; but, beyond the challenging of the writer's principal conclusion that twin jetties and dredging are about the only methods adapted to United States harbors, his claims and assertions can be stated about as follows:

* American Philosophical Society, Vol. XL, 1901.

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1.—That since about 1888 he has been endeavoring to secure the adoption of a theory or plan of bar improvement involving only a single jetty or "breakwater," which would only cost about one-half as much as twin jetties, and which would not push the bar seaward.

2.—That his plan and proposals have been pigeon-holed, for some sinister reason, to the great financial loss of the Government of the United States.

3.—That his theory has been tested at Aransas Pass., Tex., and its correctness proven.

These are very sweeping claims. To determine their truth or falsity, the writer has gone very carefully into Professor Haupt's written statements of his theories. Having twice visited and studied the alleged application of these theories at Aransas Pass, the writer is also personally familiar with that work.

The subject of our harbor bar improvement is one of enormous importance, and if Professor Haupt has a plan or theory that will save any great percentage of the heavy expenditures now considered necessary, such a plan is worthy of the fullest discussion and of the fullest consideration by the Engineering Profession.

As he does not state his theory clearly in his discussion of the writer's paper, it will be necessary to consider his other published writings on the subject. These are very numerous, comprising not less than seventeen documents, besides discussions of other papers before various engineering societies. To review all these, much of the matter being repetition, would lengthen the subject to a tiresome extent, and it will be necessary to select only the most typical papers.

In a letter from Professor Haupt to the Chief of Engineers, U. S. Army, dated June 18th, 1902, occurs the following:

"On the 9th instant a proposition was submitted to the Honorable the Secretary of War, for the opening of the Bar at the mouth of the Columbia River, by the use of the system of Reaction Breakwaters, patented in 1888. This proposal was immediately referred to your office for an opinion as to its expediency and I have, therefore, the honor of inviting your attention to a few features concerning its introduction.

"It was submitted in 1887 to the American Philosophical Society under a *nom de plume* to secure a thorough, impartial and searching examination as to its novelty and practicability. After nearly a year's consideration it was awarded the highest premium of that conservative body. It was next presented to the Board of Engineers in New York which reported against it on the ground that it was 'not new and was purely theoretical' and in consequence applications were made to your predecessors to permit a trial to be made to demonstrate its efficiency, but without eliciting a reply."

This paper, which, as stated in the above quotation, won the "Magellanic Premium" from the American Philosophical Society in

Maj. Gillette. 1888 and which should, therefore, be typical of the system, is worthy of a review of its essential portions. The paper itself is too long to quote bodily, but the quotations given are, as far as the writer can understand, full and fair exponents of what Professor Haupt meant to say, and they would not be modified in any essential particular by the addition of the context. The paper is entitled: "The Physical Phenomena of Harbor Entrances." It was referred to a Board of Engineers in 1888, and the plan it contained was reported upon adversely. Professor Haupt published a reply, entitled: "Dynamic Action of the Ocean in Building Bars." Salient and typical quotations from this paper and from Professor Haupt's other numerous writings on the subject will also be given, with references to the documents from which quoted.

The original paper, "Physical Phenomena," etc., is described on its face as a "consideration of the causes producing the typical forms characterizing alluvial coast * * *." It appears to be intended as a complete discussion of these causes, but it recognizes only one cause as important, *viz.*, the action of the flood tide. Most of his statements of fact and deductions therefrom appear to the writer to be entirely erroneous, and take no cognizance of other causes which are now generally accepted as far more potent factors than the one which he indicates as all-important. He says, speaking of the Great Bay extending from Cape Florida to Cape Hatteras:

"By following the coast northwardly from Cape Florida, it will be found that the height of the tide increases from 1.5 to about 7.4 feet at Jekyl Island, between St. Simon and St. Andrew's Sounds, which is the most remote point, about two hundred (200) miles, from the chord of the arc; also, that the outer ends of the main or ebb channels are flexed northwardly, * * *."

The fundamental fact here stated, and upon which idea the paper is principally based, that the "outer ends of the main or ebb channels are flexed northwardly," is not true. To show this, beginning at Cape Florida, the following are the principal entrances and the direction of the flexure of the main channels:

Entrance.	*Direction of the Flexure.*
Cape Florida Entrance......	Southerly.
Bear's Cut..................	Southerly.
Norris Cut.................	Southerly.
New River Inlet............	Southerly.
Hillsboro Inlet..............	Southerly.
Lake Worth Inlet...........	Southerly.
Jupiter Inlet................	Southerly.
St. Lucie Inlet..............	Doubtful, rocky.

* "Physical Phenomena of Harbor Entrances," p. 10.

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Entrance.	*Direction of the Flexure.*
Indian River Inlet	Southerly, badly defined.
Mosquito Inlet	Southerly.
Matanzas Inlet	Northerly.
St. Augustine Inlet	Northerly.
Mouth St. John's River	Southerly.
Fort George Inlet	Southerly.
Nassau Sound	Southerly.
Cumberland Sound	Southerly.
St. Andrew's Sound	Southerly.

It thus appears that this fundamental observed fact, as stated by Professor Haupt, is directly the opposite of the truth; and the principle that he illustrates by it, that the cause of the sand movement along this part of the coast is due to the inrush of the tidal wave striking the extremities of the Great Southern Bay (Cape Hatteras and Cape Florida), and then rushing in both directions toward the bight, near Jekyl Island, moving the sand to the north in the southern half of the bay and to the south in the northern half, and bending the channels across the bars of the inlets correspondingly, is wrong for the southern half of the bay. The fact is that both the sand movement and the flexing of the channels are to the south on the southern portion of the bay, which seems to destroy the foundation upon which his theory rests. His statement of it is given in the next quotation:

"An examination of our coast line reveals some striking and definite features. These are, the existence of four (4) prominent salients upon which the tidal crest impinges, and by which it is broken up into components, which are deflected into the bays on either side. At the points of incidence there will generally be found large inner sounds, extensive shoals and bars, and the heavy precipitation resulting from the checking of the momentum of the wave, the change in its direction and the interference and eddies produced thereby. Then follows the comparatively smooth reach of straight beach, along which the component tidal waves travel inland from the chord joining the salient capes, and finally, the indented and serrated shore where the opposing components in the same sinus meet at the point farthest from the chord, and where the tides are highest, the marshes most extensive, and the outlying cordon of sand is replaced by numerous islands and intricate 'back' channels. Here the tidal wave is brought to rest, and exerts its energy in a direction nearly normal to the coast, whilst along the flanks of the bay it is moving obliquely to the shore, but always towards the bight, except when locally disturbed, and with a dynamic energy, begotten in mid ocean, which compresses the sand upon the shores and transports it *in the direction of that motion.*"*

* "Physical Phenomena of Harbor Entrances," p. 9.

Maj. Gillette. At the present day, persons familiar with the subject do not dispute the fact that the tidal wave, although vastly longer than ordinary wind-waves and, in deep water, of very insignificant height, may act, to a certain extent, like them when it strikes the coast line obliquely, and will, undoubtedly, to some extent, carry with it the sand or other material which may happen to be suspended in the water near the shores which it affects, but the idea that the "large inner sounds," "extensive shoals and bars," "at the point of incidence," *i. e.*, Cape Hatteras, are due to the "heavy precipitation resulting from the checking of the momentum of the wave," etc., does not appear to be borne out by the facts. The tidal wave in the ocean being of insignificant height can be expected to affect the transportation of materials only to an insignificant depth, and there is absolutely nothing to indicate that the long shoal upon which Cape Hatteras is located, or the shoals in its vicinity, are formed by the "heavy precipitation" of the materials carried, as Professor Haupt asserts, by the flood wave approaching from the deep water of the ocean. A far more rational explanation of this is given by N. S. Shaler, in "The Geological History of Harbors,"* as follows:

"The lagoon bar element in our shore-line topography is so important, both from the point of view of science and of that of economics, that the reader should attain a clear understanding as to the manner in which these bars are formed. We shall therefore examine in a somewhat detailed way the process of construction. Wherever these reefs abound next the coast we find on examining charts of soundings which depict the shape of the bottom of the neighboring sea that the coast is bordered by a wide belt of shallow water which extends as a gradually inclined plane, declining toward the open ocean with a descent of from 5 to 10 feet to the mile, its surface covered with tolerably fine sand, mingled with the debris accumulated by the marine life which inhabits the ocean floor. From a line commonly lying from 50 to 100 miles from the coast, this broad, gentle sloping continental shelf suddenly declines into deep water, its outer margin often having a slope of 100 feet deep or more to the mile. It is clearly recognized by geologists that this continental shelf is in the main made up of debris worn from the land which has been distributed over the sea floor by the action of currents and waves, operating through a number of geologic ages, during which the shore, although occasionally rising and sinking in slight oscillations, has maintained nearly its present position.

"Wherever this continental shelf is well developed beneath the sea we are likely to find that a portion of the terrace built during periods when the coast was somewhat lower than at present extends inland in the form of broad, slightly rolling, sandy plains. Such an emerged portion of the continental shelf borders the shore from near New York to near the Rio Grande. Similar areas of recently emerged shallow sea bottom occur on all the extended sea-

* "Thirteenth Annual Report, United States Geological Survey," 1891–93, Part II, pp. 121–124.

coasts of the world, though they are perhaps nowhere else so well exhibited as in the southern seaboard states of this country, where the present coast line happens to lie near the middle point in the slope of the continental shelf. Maj. Gillette.

"As the form and structure of this continental shelf clearly indicates that the materials have been arranged by wave action, we can readily understand how portions of the material may be thrust against the shore by the heavier waves which run from the deep sea toward the coast line. It is important, however, to perceive in just what manner the wave does this work. We should first note the fact that in the deeper parts of the sea a wave of the first magnitude, though it may have a height of as much as 50 feet from trough to crest, is essentially a superficial movement of the waters in which the particles of the fluid do not go forward in any appreciable degree, but merely revolve in a kind of orbital movement. The wave motion which we make in shaking a carpet is in all essential respects comparable to that of an ocean surge where the water beneath its base is a mile or more in depth. Much as the surface of the ocean is heaved and tossed by these waves, the amount of movement imparted to the water is slight. If we could observe what takes place on the sea floor, 5,000 feet below the tempest-swept surface, it would require instruments of exceeding delicacy to indicate the trifling motions which the waves produce.

"As the waves from the deeper seas attain the shallow water next the shore—say, when they come where the sea has a depth of 500 feet—they begin to have a sensible effect upon the bottom, operating to brush the finer materials in the direction in which the surges are moving; attaining yet shallower water, this rubbing action is proportionately increased. At the depth of a hundred feet the effect of these waves in sweeping sands in toward the shore may be considerable. The action is probably of sufficient energy to drive even small pebbles up the slope toward the land. Owing to the friction which the front part of the wave encounters beyond that which the following part meets as it passes over the upward slope of the bottom, the surge becomes ever narrower in cross section as it approaches the shore—that is to say, it is higher in proportion to its width. As this friction of the bottom of the wave on the floor of the sea increases, the upper part of the surge, for the reason that the fluid there is less hindered in its forward movement by the resistance of the bottom, shoots forward, quickly acquires a wall-like front, and finally its upper part flies clear beyond the base and combs over in the form of a 'roller.' Owing to the long-continued friction on the bottom which the wave has encountered in its movement over the shallows towards the shore, its volume and energy are commonly very much reduced from the conditions presented in the open sea. Usually the surge when it breaks upon the shore has not more than the fourth to the tenth of the power which it had out in the water a thousand feet deep. Were it not for this loss of energy the effect of the ocean surges on the land would be vastly greater than it usually is.

"Watching the action of ocean waves along a gently sloping shore, we observe that the lesser undulations, such as occur when the

Maj. Gillette. sea is affected only by inshore winds of slight energy, break very near the water line. Heavier surges—say, those having a height of three or four feet—comb and fall over at a distance of some scores of feet from the actual margin of the sea, while waves of the greatest volume, such as are formed at rare times of great tempests, may break a mile or more away from the strand. Wherever the overturning occurs the power of the wave is broken and whatever debris it may have been urging forward is left upon the bottom. If we clearly perceive these features in the action of the waves it is easy to understand how the bar islands which inclose lagoons are formed. Thus in case our southern shore should be depressed below the level of the sea, so that the barrier sand reefs were covered, a result which would be produced if the region were lowered to the depth of thirty or forty feet below its present level, the immediate effect would be to bring the ocean waters into free contact with the shore of the mainland in a manner found on coast lines where there are no such outlying islands. At once the submerged barrier would be taken to pieces by the waves and the accumulations of sand would be spread over the bottom of the sea, still further shallowing the water next the shore. With the advent of the next ensuing great storm the waves would break at a distance from the shore; it might be even some miles away from it and on this new line of breakers the construction of a new series of barrier sand reefs would begin. If the storm was great, so that the waves were of the first magnitude, the breaking might take place in water having a depth of as much as 50 feet. Waves of a volume to break in water of this depth would carry a good deal of sand to the point where they topple over. Here this transported detritus would lodge, and if the storm were long continued the sands might be built up to a sufficient height for the ridge to emerge above the level of the sea and form a beach. Unless this emergence of the crest were effected, the succeeding storms of lesser energy would tend to destroy the imperfect barrier by sweeping waves over its crest without breaking, in which case, as will be readily perceived without further explanation, they would tend gradually to scour away the elevation, distributing the sand between its position and the neighboring shore. The shallowing of the water thus brought about might go on by the successive temporary formation and distribution of submarine sand reefs, until finally, in some great storm, a ridge was built up of considerable length, perhaps along a great distance of shore, which rose above the level of ordinary low tide. When such a ridge had been so formed, high enough to escape the scouring action of waves of any considerable magnitude, operating by overrunning its top, each succeeding storm, even those of ordinary energy would tend to add to the mass by bringing in more sand from the continental shelf.

"As will be noted hereafter, the inner portion of the continental shelf, the part which lies in shallow water next the shore, commonly receives considerable contributions of sand, which work along the coast from regions beyond the limit of the barrier reefs. Thus on our Atlantic coast a good deal of arenaceous material may have journeyed from as far north as New Jersey, where it was contributed to the sea during the last glacial period, or washed into the ocean

from deposits of sandy matter formed during the ice time. In this way the waves can continually bring in sandy matter without diminishing the depth of water on the outlying shallows. So far as depends upon the action of the waves the coastal sand islands can not rise more than a few feet above the level of low tide, but as soon as a beach is formed the winds operate to form sand heaps or dunes above the level of the sea which may considerably increase its elevation above the ocean level." Maj. Gillette.

From the foregoing quotation it is plain that the exposure of Cape Hatteras to the heavy wind waves rolling in from the ocean has enabled them to beat up the sands before them and to build the long line of narrow sand islands upon which the cape is located. This action can be understood with wind waves, which affect the bottom to great depths, and stir up the sand, but the formation is wholly unexplained by Professor Haupt's "heavy precipitation" from the checking of the tidal wave, which, at this point, only obtains an elevation of 2½ ft., and takes 6 hours to rise to this height. Furthermore, at the Cape Florida extremity of the Great Southern Bay the tidal wave is about the same as at Hatteras, but there is no corresponding heaping up of the sand. This is readily explained by the difference in the exposure to storms, the southern waters being habitually smooth and quiet, while at Hatteras storm waves and rollers are breaking nearly all the time. The sand cordon at Hatteras is of precisely the same origin as that along the shores of the Carolinas, Georgia and Florida. Its greater distance offshore at Hatteras is explained by the greater exposure and probably by originally shoaler water.

"This reach of coast is characterized by three secondary bays, separated by the groins of Cape Lookout and Cape Fear. These capes are the resultants of the opposition of the tidal wave to the fresh water discharge, which being unable to effect its escape in the face of the flood is turned to the west and south by the pressure of the tidal component deflected from Cape Hatteras."*

This reasoning might possibly be true of Cape Fear, which has a fresh-water river flowing out at the point, but Cape Lookout, which is precisely like it in other respects, has no fresh-water flow, and hence its formation must be explained in some other way. A much more rational explanation† indicates that the cusps enclosing these three bays are the results of eddy currents from the Gulf Stream, *viz.:*

"Form: The four great capes along the eastern coast of the United States, namely, Hatteras, Lookout, Fear and Canaveral, are so well known and have been so frequently mapped that a general description is here unnecessary. The theory of current cusp formation will therefore first be considered and then more detailed facts of form introduced afterwards.

* "Physical Phenomena of Harbor Entrances," p. 19.

† "Cuspate Forelands," by F. P. Gulliver, *Bulletin* of the Geological Society of America, Vol. 7, p. 402, *et seq.*

Maj. Gillette.

"Backset Eddies.

"The ocean circulation is made up of great eddies, which in turn set up smaller eddies between the main current and the coastal border. These smaller currents revolve in the reverse direction to that of the great circulation. Eddies of this kind are appropriately called backset eddies. When more waste is supplied than the on and off-shore components of the total sea action can spread over the bottom, the alongshore component will deposit it where it encounters dead water or water moving with less velocity than the current itself. In a backset eddy system of circulation such comparatively dead water will occur where the alongshore currents curve from the shore toward the deep sea. Slight inequalities of outline in the newly born land may break the backset current into eddies of varying radius of curvature. Between such eddies there would be formed a triangular space of comparatively dead water, in which the growth of a cusp might be expected. Given the backset eddies and the inequalities or even a straight shore line, and cuspate forelands at least in outline could be formed, for a detritus-laden current must deposit the surplus of its load along its margin where it comes in contact with quiet water.

"Combination of Currents.

"There are three possible pairs of currents which might produce cuspate forelands upon the outer shore line. First, both currents flow toward the land; second, both currents flow toward the water; third, one current flows toward the land and the other toward the water.

* * * * * *

"According to the third scheme, a current flows toward the land upon one side of the foreland, while upon the other side a current flows toward the sea. As has been pointed out, such an arrangement would result if a dominant current alongshore were broken by projections of the land into several eddies.

Such a system of backset eddies has been suggested by Mr. C. Abbe, Jr., as the cause of the great Carolina cusps, Capes Hatteras, Lookout and Fear. Such currents seem to be proved by observations along the shore."

The above clearly accounts for the existence of the three bays in question.

Professor Haupt's theory should call for similar cusps at the mouth of the Santee, Ashley and Edisto Rivers, as there should exist at the mouths of these streams the same "opposition of the tidal wave to the fresh-water discharge," and similar capes should be formed at each one of them, whereas not one exists. On the other hand, the above "backset eddy" theory called for such formation only where the conformation, etc., is such as to cause an eddy, and it has nothing to do with the fresh-water flow. The detritus brought down by the rivers and deposited at their mouths, forming shoals, may, of

course, have been one of the incidents that helped to determine the location of the "backset eddies." Maj. Gillette.

Professor Haupt says:

"It is also a notable fact that a straight line drawn from Cape Romain to Hatteras is just tangent to Capes Lookout and Fear, and that the transverse and semi-conjugate axes of the ellipses of Long and Onslow Bays are respectively equal, while Raleigh Bay is somewhat smaller in both directions and has a steeper scarp than either of the others (due to the incident wave). The shore to the north of Cape Hatteras is deflected from the chord of the three bays produced at an angle of 45° for a distance of twenty-six (26) miles, when it again bends to the westward through an angle of 30°, and continues in an unbroken stretch of ninety-four (94) miles to Cape Henry. The only outlets on the eastern cordon of Hatteras are near the point of deflection where the northward component from the cape and the normal wave commingle.

"The cotidal curve of eleven and one-half (11 1/2) hour interval envelopes the cape in an arc of a circle whose radius is seventy-three (73) miles, whilst the shore line changes its direction through an angle of 74°. The limiting radii of this sector also pass through the main openings of the cordon at Oregon and Ocracoke Inlets, which are opposite the tangent points of the arc, and hence indicate the locus of the change of direction and weakening of the tidal wave. The coast line will also be found to be inclined to these radii at an angle of 80° which indicates the direction of the shore component at the points of intersection. At Ocracoke the chord of the bar lies 10° north of the tangent, and at Oregon * * * 10° to the west, showing the movements to be east and north.

"The velocity of the wave is greatest along the normal at Cape Hatteras and least along the radii limiting the sector. An examination of the interval between the cotidal lines shows also that the rate of movement of the general crest is considerably retarded as it approaches the shore. The twelve (12) hour crest will be seen with its flanks rolling along the receding shores of the bays, as already described. The earliest points of contact of the tidal wave are readily discovered from this chart to be at or near the points formerly indicated in this paper, whilst the latest point is at Jekyl's Island, which is found by measurement to be just midway between Capes Florida and Hatteras, thus making the times of transit to this point of meeting of the flood components, equal."*

The first part of this rather elaborate mathematical discussion is somewhat vague as to its connection with the subject. The latter is based upon fine measurements of slight curves of some cotidal lines made many years ago and concerning which the data were then, and are now, extremely uncertain. It will perhaps be sufficient to state that more recent cotidal lines in that vicinity, based on more extended observations, worked out for the writer by the present officials of the United States Coast and Geodetic Survey, do

* "Physical Phenomena, etc.," p. 11.

Maj. Gillette. not show anything of the kind. According to these more recent, and presumably much more reliable, cotidal lines, the curve of 11½-hour interval strikes Cape Lookout considerably in advance of Cape Hatteras, makes a slight curvature about 9 miles off Cape Hatteras, and then strikes in a straight line directly northeast. There are no tangent points of the arc, "opposite Oregon and Ocracoke Inlets," and the present position of the more recent lines makes the quoted mathematical discussion wholly irrelevant. When it is remembered that the position of the curve of 11½-hour interval might change its position 50 miles in a few minutes, by a trifling change of a few inches in the height of the water surface, and that its position could not, by any human appliances, be measured in place, Professor Haupt's painstaking deductions from its accurate measurement on the ancient maps appears to be wasted energy.

Incidentally, it may be remarked that Jekyl Island, instead of being "just midway," is about 100 miles nearer Cape Florida than it is to Cape Hatteras, and that, according to the present cotidal lines, high-water mark is reached at Cape Lookout about 1½ hours before it is reached at Cape Canaveral, and that Jekyl Island, instead of being the last point reached by high water, is also ¼ to ½ hour ahead of Cape Canaveral and is ahead of the entire shore between those two points.

As to the relative importance of the tidal approach and of waves and currents caused by the wind, the following appear to be the facts: Waves, breaking diagonally upon a sandy beach, stir up and put in suspension for a time the finer particles of sand, and, at the same time, by the shock against the shore, move forward in a diagonal direction particles too heavy to be carried in suspension. The greater the waves, the greater the force, and if the total amount of this wave action in one direction is greater than in the other for long periods, there is bound to be a resultant sand travel in the corresponding direction. If the waves break for long periods in one direction there is also bound to be an accumulative motion of translation causing a definite current in that direction, which current may include the water for a considerable distance offshore.

The transportation of the particles in suspension may be aided or opposed by any currents that affect the vicinity (for example, great ocean currents, like the Gulf Stream), or eddies from them.

The tidal wave in the open ocean produces practically no current. The amount of current produced near the shore is determined principally by the conformation. When the tidal wave goes into a wedge-shaped bay, like the Bay of Fundy, no one disputes the fact that strong currents may be caused, capable of eroding sand or other materials and carrying them with it, but when such a "bay" is 700 miles wide at the mouth and of very gentle curvature, as in the

case of the Great Southern Bay, the current generated must be very small, particularly if, as in this case, the crest of the tidal wave strikes the bottom of the bay curve before it does any other part of its southern half, as has been shown above. Littoral drift along the general eastern coast line of the United States, therefore, can be only very slightly affected, if at all, by any current produced by the "dynamic energy" of the flood "begotten in mid-ocean, which compresses the sand upon the shores and transports it in the direction of that motion." Maj. Gillette.

Subsequently, in the paper entitled "Dynamic Action of the Ocean in Building Bars," Professor Haupt makes somewhat of a "frank change of front" and admits, in the form of a claim, that something else besides the current of the flood tide or the velocity of the tidal wave does the work. He says:*

"In presenting the evidence in reply to this Report (Board of 1888), I propose to show:

"(1) That the *velocity* is an unimportant factor, and that material can be transported even where there is no motion of translation in the motor.

"(2) That waves breaking obliquely on a sandy shore will move the particles over a zigzag path, in a constant direction, which is cumulative.

"(3) That the flood tide produces such angular waves, and that littoral currents aid the movement.

"(4) That the term *flood component* is more comprehensive than flood *current,* and includes the dynamic action of the breakers racing along the shore, as well as the littoral currents generated by the onshore movement of the flood tide."

As far as the writer has been able to ascertain, however, he gives no information as to how "the flood tide produces such angular waves," and, while he wishes to include in his term "flood component" the "dynamic action of the breakers racing along the shore," he gives no evidence whatever to show that such breakers are ever produced in any way by the tide.

The term "flood component" is a very indefinite one. Ordinarily, a component is one of the parts of a force or movement, determined by direction. Thus the littoral or alongshore component of the flood means something—*viz.,* one of the currents into which the flood movement may be resolved; but, what other components taken with the flood component make the whole, and what is that whole? There is no such entity unless it might be the total tidal movement—the other component being the ebb component. But this is not what Professor Haupt means. Apparently, after the Board of Engineers in 1888 showed that this littoral component of the flood was an insignificant factor, and that he had omitted all mention of the more important influence of the wind waves, he gives a

* "Dynamic Action, etc.," pp. 2 and 3.

Maj. Gillette. new meaning to the illogical expression "flood component," and includes also wave action in it, but fails to demonstrate, as he claims, that such waves are ever created by the flood tide.

In this connection he further says:*

"If it can be shown that the *flood currents have* sufficient energy to move materials, such as *bricks, coal, wreckage.* etc., in a *direction opposed to the winds,* even during storms, and for distances measured by miles *in the direction of the flood,* it would seem to be sufficient evidence to prove not only the *existence* of such a force, but that it is 'sufficiently powerful' to move *sand* beneath the surface in the same direction."

In proof of this he gives examples of bricks, coal and other materials moved along the western part of the south coast of Long Island toward the west, and along the coast of New Jersey near Sandy Hook toward the north, both in accordance with the undoubted direction of the tidal movement. He does not give such examples for any other places. This is worth looking into.

An examination of the coast line will show that the western portion of the southern coast of Long Island and the northern portion of the coast of New Jersey constitute a large, rather wide-mouthed bay, and that a pretty strong current can be expected in this vicinity toward New York Harbor, from the inflowing tide, as in any bay, which current must have its influence in moving materials in suspension. But this alone is far from sufficient to account for the steady movement, toward New York, of bricks, coal and other moderately heavy materials deposited on the beaches. Such materials, however, are readily moved by the shock of the waves, and it has not yet been demonstrated that waves capable of producing such shocks are generated by anything but the wind. From the simple fact that the offshore winds have no particular effect in this connection and that winds to produce high waves must have a considerable "fetch," it is easily seen that a wind to move material to the east on the part of the Long Island shore under consideration must blow from the southwest over New Jersey, and thus, having only a short travel across water, it would be ineffective; while a wind to move material to the south on this part of the New Jersey coast must come from the northeast and be masked in a precisely similar manner by Long Island. It is, therefore, easily seen why the only effective winds which can blow in this vicinity must be from the east and south, and the waves produced by these winds would all move such materials toward New York. These are abundant to account for the movement of the material, without any assistance from the admittedly strong tidal currents, which do flow toward New York on account of the configuration above mentioned, as well as on account of the fact that a large quantity of tidal water

* "Dynamic Action, etc.," p. 3.

probably goes into the southern entrance of New York Harbor and goes out to the east through Long Island Sound. Maj. Gillette.

These facts explain the fallacy of the following quotation from the same paper:*

"The wind-wave theory is totally inadequate to explain the existence of the peculiar hooks and spits which have been built out directly in the face of the prevailing winds. For instance * * * Sandy Hook."

On page 6 of the same document he says:

"But as a matter of fact the resultant sand movement is southwesterly, or in a direction *opposed* to the prevailing wind; so that this theory (wind wave) is untenable in almost, if not in every instance."

And:

"The wind-wave theory also fails signally as applied to the Great Lakes."

As no one familiar with the subject now doubts the all-important potency of wind waves in producing littoral sand drift, it is not probable that Professor Haupt would now reiterate this generalization.

The above include the principal quotations from Professor Haupt's writings, in his endeavor to prove that a littoral drift along the general coast line is caused by the flood tidal wave. From the facts given, it appears to be entirely fallacious, as applied to the Atlantic coast of the United States, and that the tidal wave is of very minor importance in controlling littoral drift on this coast, except possibly in the last case, like the bay above mentioned near New York.

From the fact that there is marked littoral drift in the Gulf of Mexico, where the tides proper are insignificant, in the Mediterranean, where there are practically no tides, and in the Great Lakes, where the wind waves are frequently from 13 to 14 ft. high, and where it took years of careful observation to establish a probable tide of a fraction of an inch,† it would appear that Professor Haupt's theory of the flood tide being the principal cause of littoral drift is wholly untenable.

W. H. Wheeler, M. Inst. C. E., in 1896, presented, before the Institution of Civil Engineers, a very elaborate paper‡ on littoral drift. It was discussed orally and in writing by thirty-five other members of this eminent institution, many of them giving examples from their own experience. An analysis of all the cases mentioned

* "Dynamic Action, etc.," p. 5.

† The Great Lakes have occasional fluctuations of a few inches, on rare occasions amounting to 2 or 3 ft. These are called "seiches." The little ones appear to be caused by barometric changes, the larger ones by the wind. They are quite irregular in direction, and cumulative results cannot be expected from them.

‡ *Minutes of Proceedings*, Inst. C. E., Vol. CXXV.

Maj. Gillette. indicates that, while the direction of the flood-tide approach often coincides with the littoral drift, it is a fact that whenever there are strong onshore winds, with a marked predominance in one direction, the littoral drift goes with that wind, irrespective of the direction of the flood tide, if there be a tide.

Forces Causing Ocean Bars.

Professor Haupt states:

"I have heretofore elsewhere called attention to the important deductions to be obtained from noting the position of the submerged crest line of bars, as well as from the relative slopes of sections along the thalweg of the channel or across the bar, as indicating the direction of movement of the sand and of the flexure of the outer ends of the channels, after passing the gorge, either up or down the coast. The immediate cause of this flexure was asserted to be a littoral component which *rolled up the sand on the flood tide and compressed the ebb stream against one or other of the adjacent shores.*"*

It is not disputed that the flood tide produces currents, and that these currents will carry with them the suspended matter which the water happens to contain, or, if of sufficient velocity, they will scour and carry with them materials corresponding to that velocity, and that such currents should be given proper consideration in any particular problem, but the foregoing quotation, and, in fact, the entire pamphlet, gives to the flood tide the whole credit for sand movement up and down the coast, omitting entirely the action of wind waves, which are now generally accepted as having a vastly more important effect. The author appears to have taken no cognizance of the now well-recognized cyclical movement of the main channel across an ocean bar, by which it is gradually driven by the sand drift in one direction until it parallels the coast and then breaks out, usually in a much better and deeper channel, more nearly straight to the sea. He certainly makes no mention whatever in this paper of this most important fact concerning the problem he has under consideration.

He says:

"An examination of the various entrances leaves no doubt of the existence of such a littoral flood movement, whereby the sands of the beaches are transported to and deposited in front of the inlets, where the racing waves, no longer resisted by and reflected from the shore, escape through the break in the barrier which forms the outlying sandy cordon defending the coast.

"The effect of this racing of the waves in search of an escape from the pressure of the flood tide is to scour off and prolong the sharper lip at the gorge and to flatten out and beat back the opposite

* "Physical Phenomena of Harbor Entrances," pp. 1 and 2.

shore, thus shifting the position of the 'inlet' until in some instances it is transported considerably to one side of the medial line of the inner bay, or entirely closed. Thus, the position of the thalweg is made to cross the gorge oliquely, and furnishes additional evidence of the resultant direction of the external or flood movement."* Maj. Gillette.

No one doubts the existence of littoral sand drift, but it is not clearly understood why waves should race in search of an escape from the pressure of a rising tide. Professor Haupt claims that these waves are the offspring of the flood tide, and the above paragraph would indicate a desire to escape from their parent, but neither the method of their origin nor the cause of the desire are explained. Of course, ordinary wind waves beating along the shore will, at an inlet, pass freely in and be dissipated, but there is nothing in this to indicate that such waves are produced by the flood tide. Generally, the littoral drift, reaching an inlet, prolongs one side of it, and, by causing erosion of the other, makes the inlet travel with the littoral drift, but this is by no means always the case. For example, the sand drift on the coast of Georgia is distinctly to the south, yet the inlet at the lower end of Blackbeard Island has, within the last few years, moved northward more than a mile, somewhat more than a half of which was by breaking out, and the remainder, about 1 800 ft., has been by steady travel northward, which travel has caused the Marine Hospital Buildings to be moved once, and the indications now are that a second move will be necessary. This is shown on Fig. 34.

In another place,† Professor Haupt speaks of certain harbor works as being "so designed as to divert the ebb stream directly into the face of the flood, where the resistance to be overcome is the greatest." These things would indicate that the escaping ebb tide has to struggle against the incoming flood, but as the ebb outflow occupies a period of 6 hours and the flood inflow another 6 hours, such struggle could only take place at the unimportant periods of nearly slack water, and it is difficult to understand just what the "flood resistance" and the "face of the flood" have to do with the ebb flow.

Also,‡ referring to ocean bars:

"The *external* effects are those resulting from the form, position and extent of the banks which have been piled up by the flood," etc.

Also:§ "This enceinte formed by the flood," etc.; and,‖ referring to the flood tide as a bar builder:

"What I did claim and emphasize in my paper was not that, but their efficiency and controlling influence as bar-building agencies,

* "Physical Phenomena of Harbor Entrances," pp. 5 and 6.

† "Physical Phenomena, etc.," p. 15.

‡ "Physical Phenomena, etc.," p. 4.

§ "Physical Phenomena, etc.," p. 7.

‖ "Dynamic Action, etc.," p. 11.

Maj. Gillette. and I applied the knowledge of the direction of the flood component to the designing of a plan for successfully resisting these encroachments.

* * * * * *

"I think it is clearly demonstrated that there is a flood component of greater or lesser intensity, depending on the angle at which the flood movement breaks upon the shore, and that it is the

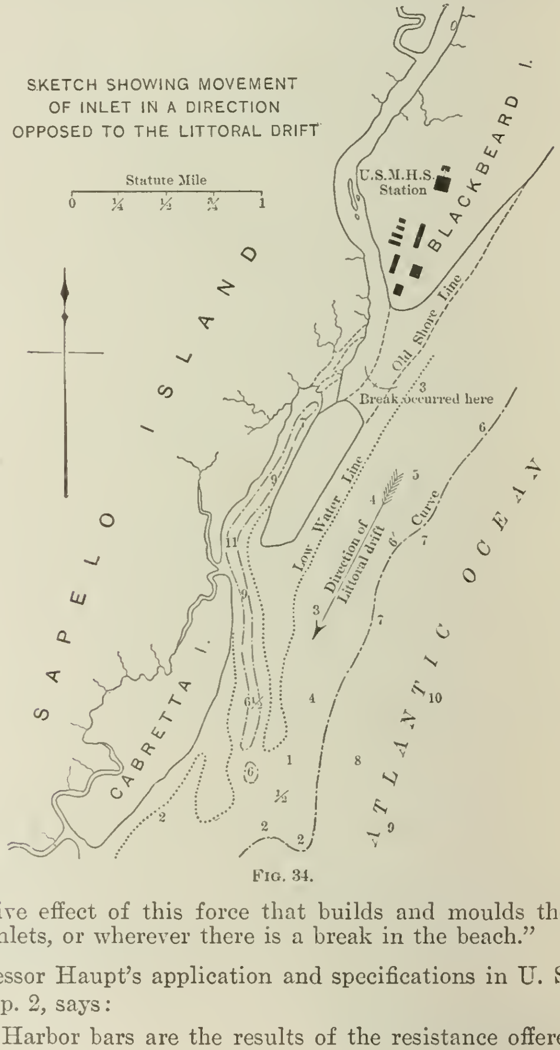

Fig. 34.

cumulative effect of this force that builds and moulds the bars at harbor inlets, or wherever there is a break in the beach."

Professor Haupt's application and specifications in U. S. patent, 380 589, p. 2, says:

"(c) Harbor bars are the results of the resistance offered by the shores to the momentum of the flood tide."

This claim that the flood tide builds the bar in front of an entrance seems to be totally refuted by the fact that in seas and lakes where there is an ebb flow, due to a river, but no flood flow at all, as in the Mediterranean, or only occasional "seiches," as in the Great Lakes, the phenomenon of bar formation is precisely the same as at the mouths of rivers opening on oceans with heavy tides, and indicates that the flood tide of itself has a very minor part to play in bar formation. If there were no outflow through the gaps in the cordon of sand islands along the South Atlantic coast, the wind and waves would very soon close every one of them by driving the bar in toward the gorge until it rose above the water's surface. This force is ever present where there are onshore winds. Offshore winds have no effect, except to cause quiet water in which sand may settle along the shore. The force which opposes the wind waves in their efforts to drive the bar in is the outgoing flow, whether of tidal water or land drainage. The variations of the bar are due to the variations in these forces, complicated by the sand travel, caused by the waves, when they strike the coast diagonally, and by other sediment. All these phenomena could occur, and do occur, without any "flood component" at all. Hence, there is no apparent foundation for the claim that the flood component is a "controlling agency." Maj. Gillette.

Professor Haupt's Plan of Improvement.

"A typical plan for a breakwater which will not produce eddies or objectional shoals, nor be eaten away by the sea, would be one composed of curves whose cusps are pointed in the direction of the advancing flood resultant, and having an inshore flank to concentrate the flood upon the beach channel, where it is both possible and desirable to maintain one. The curves should have the semi-conjugate diameters equal to about one-fourth (¼) of the transverse. The interferences resulting from this *form* will produce shoals in front of the groins, thus reinforcing them, and as the outer end of the breakwater is pointed so as to receive the flood point blank, there will be no eddy nor any abrupt checking of its velocity inside to cause the shoaling, yet the flood will be freely admitted and there will be a circulation created by having the beach end open. During the ebb there is no interference with the main current, but there is a concentration of its energy upon the weaker portion of the bar. For an illustration of this plan reference is made to the location on the chart of Charleston (Fig. 1),* submitted herewith. The jetties, U. S. J., now under contract, cover a total length of six (6) miles. Those projected, of but three (3) miles, and the latter will make two (2) good channels, one for flood and one for ebb, while it is very doubtful whether the former will produce any material improvement of the entrance, but it will advance the general shore line and push the bar further to seaward."†

* Fig. 35 in this discussion.

† "Physical Phenomena of Harbor Entrances," p. 18.

Maj. Gillette. And, in the specifications of U. S. patent, 380 569 (p. 2):

"In harbors the proper form for a breakwater to secure these desiderata is one composed of a series of intersecting arcs having their cusps or salients so placed as to cut the advancing waves and resolve them into components along the concave faces of the structure which is intended to extend above high water. By this means the opposing components in the same cove will neutralize each other and the transporting power of the wave will be destroyed and shoals will form outside the barrier, which will tend to re-enforce it and establish its position.

* * * * * *

"The curves may be placed with their vertices opposite each other forming a double funnel-shaped passage of the form of an hour glass through the gorge of which the tidal currents would be compressed with increased velocity."

The diagram of the work proposed for the improvement of Charleston Harbor (Fig. 35) thus provides for the catching of the incoming tidal wave by scallop-shaped breakwaters, concave to the sea and joined at acute angles, being designed "to receive the flood point blank," and split it up into currents which neutralize each other at the bottom of the "coves." It needs no demonstration to show that these scallops would not act in a manner essentially different from a straight-line structure, especially after the small quantity of sand necessary to fill up the "coves" had been deposited, and that the storing of this quantity would not be worth the extra cost of building the works on the curved lines, let alone the additional material necessary to construct them. Such a structure has never been built, nor, as far as known, has it ever been advocated subsequent to this paper, even by Professor Haupt. Appended to the structure is a wing intended to improve the beach channel which exists in this and all similar harbors. This feature would make an admirable entrance for the littoral drift to come into the harbor. The existing government jetty is raised to only about low-water mark across this channel, and Professor Haupt, himself, has since condemned this feature many times,* because he claims that it so admits the sand. The builders of the present jetty maintain that as yet no sand has gone over it, but Professor Haupt claims that there has, which, if true, would certainly condemn the return wing of his scalloped breakwater designed to scour out this beach channel so as to admit the flood tide freely.

In a similar case, since quoted by Professor Haupt on several occasions, a short jetty was built to close up such a beach channel. This was at Aransas Pass, Texas, in 1869. This jetty, or the constituent parts of this jetty, were:

"Expected to act as a nucleus about which the sand would settle

* *Transactions*, Am. Soc. C. E., Vol. XLVI, p. 523.

Maj. Gillette.

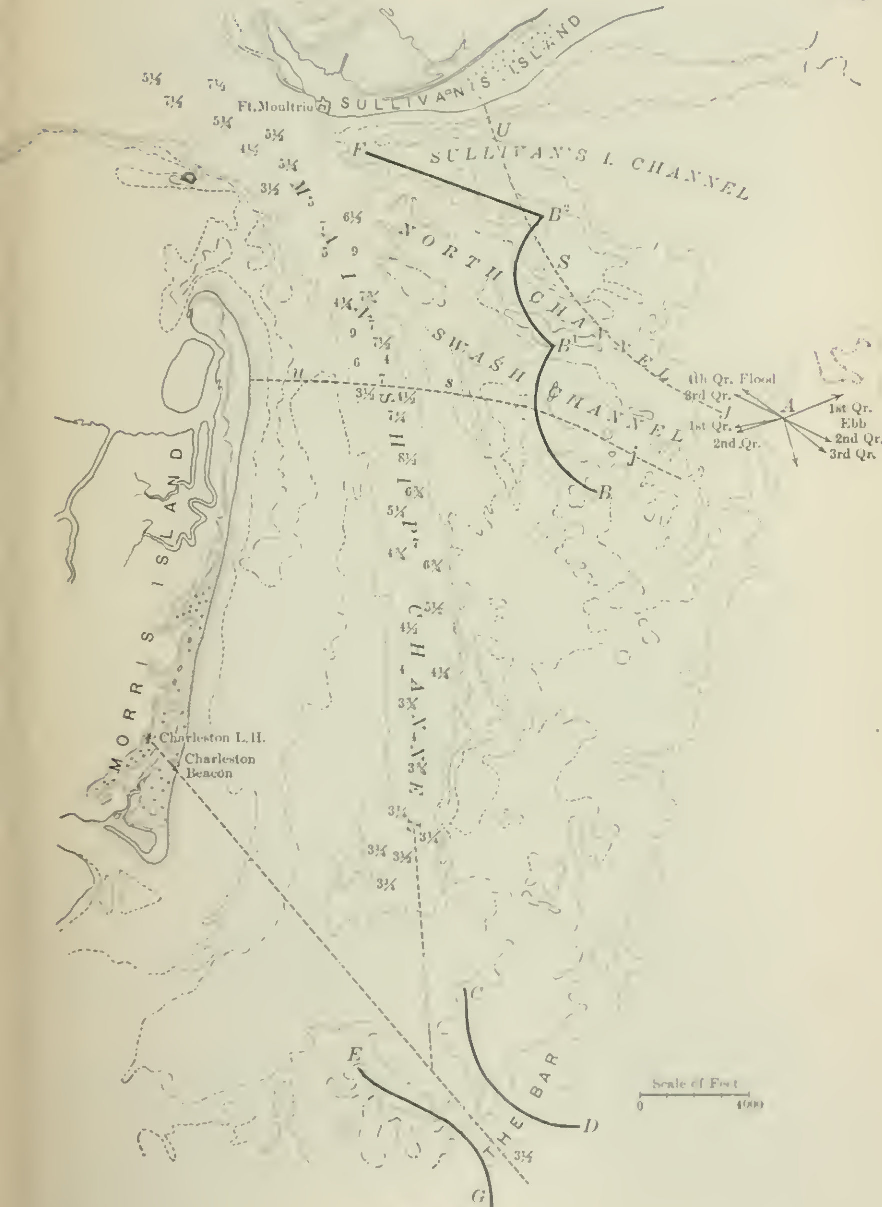

Fig. 35.

Maj. Gillette. and close up the secondary channel, thus directing the flow of water directly through the channel of the bar."

"From the fact that the secondary channel has shoaled about two feet and the main ship channel deepened about two feet since placing the crates, it may be supposed they have contributed to produce this result."*

Professor Haupt says of this:† "These extracts show very conclusively that, so far as the frail structure went, it was in the proper place * * *."

This would certainly seem to condemn the beach channel which he proposed to deepen at Charleston, as he assumed the direction of the sand drift to be the same in each case.

As to the objection which Professor Haupt made above to the twin jetties at Charleston, it may be said that while they undoubtedly have scoured seaward a large part of the mass of sand that was between them, and possibly some that has come over the low shore end of the north jetty, yet there is to-day a good channel, more than 25 ft. deep at low water, into this harbor, and the writer very much doubts if Professor Haupt would to-day advocate a single element of his design above proposed for its improvement.

His hour-glass structure, in the light of present knowledge, seems to be especially valueless. It is located more than 4 miles from the other structure—much too far to receive any assistance from it. It is to improve the bar channel when flexed to the south to its maximum extent, when the slightest further resistance to the ebb flow would cause that channel to close and a new cycle to begin, with the bar crossing miles away to the north, leaving the "hour glass" buried in a useless and uninteresting sand bank. The "compression of the currents" proposed, would furnish just the needed resistance to the ebb flow to produce these results.

It will be shown, later, that the breakwater at Aransas Pass, which Professor Haupt, in the above quoted letter to the Chief of Engineers, U. S. Army, as well as in his discussion of the writer's paper, claims was in accordance with the same ideas as were embodied in the paper on "Physical Phenomena of Harbor Entrances," is not at all in accordance with the theories therein laid down.

The claims upon which the Magellanic Prize for the above paper was asked and received are fourteen in number, and are quoted and discussed here.

1.—"The determination of the character, direction and relative intensities of the forces acting upon any harbor entrance, from a study of the submerged topography and other physical features."

This theoretical plan of study is useful, but does not cover the whole subject, taken alone. It may lead to far-reaching errors, as the above discussion demonstrates.

* Report of Chief of Engineers, 1871, p. 526.

† "Dynamic Action of the Ocean in Building Bars," p. 22.

2.—"The discovery of the existence of typical forms in the sandy spits bordering the entrance, which will in general indicate the direction of the resultant movement." Maj. Gillette.

The particular forms of sandy spit pointed out by Professor Haupt, as indicating the direction of littoral drift, cover only a small fraction of the subject. Taken alone, as was to be expected, they led him into errors. They indicated to him that the movement of the entrance coincides with the direction of the littoral drift, which is not always true, as in the instance above shown of Blackbeard Island, where the movement of the entrance is very marked, and directly opposed to a well-established littoral drift. Another example that can be given is Cumberland Sound, Georgia and Florida, where a curved marsh area, around the Town of Fernandina, has a shape decidedly similar to the channel in 1843, indicating that the channel once occupied the site of the marsh and that it moved to near its present location, about ¾ mile northward, on some occasion when the channel had been driven by the littoral drift from the north until it hugged the shore to the south so far that it was easier to break through the island to the north than to cut through the sand directly seaward, as it usually does. The sketch, Fig. 36, shows the similarity. The marsh near the southern end of Cumberland Island had to be very carefully watched while the jetty work was in progress, and some very active work had to be done there recently to prevent a repetition of this precise action. The position of the sand dunes near the head of the island and just south of the present channel certainly indicates a steady movement of the entrance northward since, or in place of, the above supposed breakout. The drift at this point is unquestionably from north to south.

As will be shown further on, the entrance at Aransas Pass, traveled to the south, while the littoral drift is to the north, the travel being caused by the driving of the water out of the inner lagoons by fierce "Northers," a type of wind almost peculiar to that coast.

Another case is Humboldt Bay Entrance, California. Here the resultant littoral drift is unquestionably from the south—yet the entrance, in the twenty-two years between 1858 and 1880, moved south not less than 1 900 ft.

3.—"The recognition of the fact that the proper place for the ebb discharge, or channel over the bar, is as far removed as may be from the point of direct attack of the flood resultant, when the direction of the latter is not normal to the coast."

This is one of that class of facts which are "important, if true." It was recognized by Professor Haupt in this paper; but, so far as known, has never been recognized since by himself or others in the design of work for the improvement of ocean bars. This would

Maj. Gillette. make the bar crossing at the point where it exists after the channel has been "flexed" to its maximum amount, which arrangement is a very poor one. This seems to be the place which the diagram in Professor Haupt's plan for Charleston indicates as the proper one to improve. The objections to it are that this point is so far removed from the gorge that the ebb discharge loses its force, by reduction of slope and by loss of volume over the long sand cordon

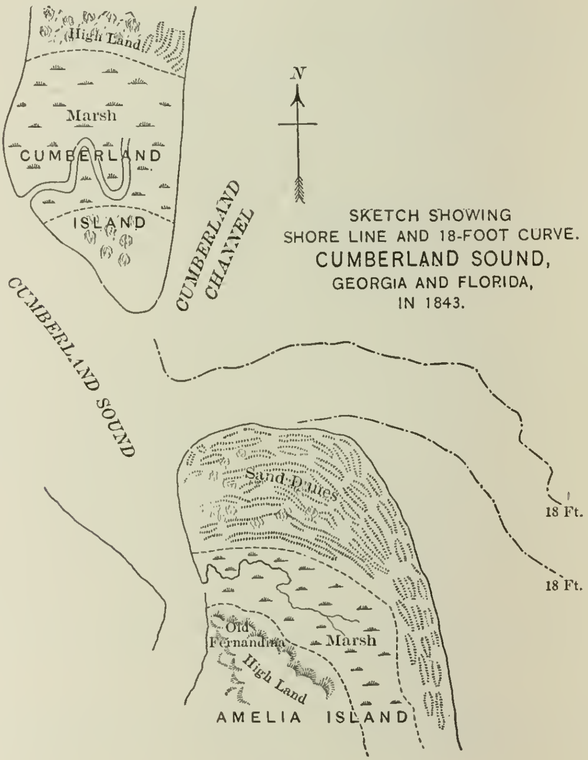

FIG. 36.

between the two points, and becomes ineffective. Further, this plan would attempt to improve the depth at a point where Nature usually provides the poorest channel and one just preparing to "go out of business."

His paper does not take cognizance of the most important matter in this connection, *viz.*, the cyclical change in such channels. The channel, just after first breaking out directly to sea, not only requires shorter works for improvement, but is usually in its deepest

natural condition. The nearer the bar crossing is to the gorge the steeper will be the slope, the more effective the ebb discharge and the more favorable the entrance for shipping. It is possible, however, that Professor Haupt's diagram of Charleston may be intended to provide for the creation of a new bar channel just south of the breakwater, as in a subsequent paper he claims: Maj. Gillette.

"As to my proposed channel being so lengthened as to fritter away the working energy due to difference of head, it is only necessary to say that the point of escape for the ebb at all these sites is, in my plans, nearer the gorge, giving a greater slope and more rapid discharge than in the plans now under construction. At Charleston, the most unfavorable case for me, it is but two and seven-eighths miles distant from the gorge, while the mouth of the Government jetties is about three and one-eighth miles distant."*

The distance, on Professor Haupt's diagram (similar to Fig. 35), from the gorge, opposite Fort Moultrie, to the point marked "the bar," is 6¼ miles. This point corresponds to the point, "*B*," of his diagram†—the point of "minimum flood resistance," which he says "is that point of the bar farthest removed from the direct action of the flood," and which, in the above claim No. 3, is the one to which he refers. His claim that it is only 2⅞ miles from the gorge, in the foregoing quotation, is, therefore, incomprehensible. In any event, his bar crossing must be at some point south of the scalloped jetty (Fig. 35), which cannot possibly avoid making it farther from the gorge than the bar crossing between the government jetties shown on the same drawing. Subsequent to his original paper, but previous to the above claim that his proposed "bar crossing" or point of escape for the ebb was only 2⅞ miles from the gorge, he applied for a patent, and his patent drawing (Fig. 35) shows his remarkable hour-glass structure added to improve the channel at the above point, which is certainly 6¼ miles from the gorge.

4.—"The definite enunciation of the principle that the trend of the coast with reference to the cotidal line will in general indicate at once the proper position for defensive works."

As shown above, to adopt the cotidal line to "indicate at once the proper position for defensive works" would be to adopt about the most vague, indefinite and uncertain guide conceivable. For example, the most of the curve of 11½-hour interval, shown on Professor Haupt's cotidal map for the Great Southern Bay of the Atlantic Coast, is almost exactly at right angles to that recently worked out by the Coast Survey, and their southern extremities are distant from each other about 300 miles. It is probable, therefore, that this general guide will never be adopted by maritime engineers to "indicate at once the proper position for defensive works."

* "Dynamic Action of the Ocean in Building Bars," p. 16.

† "Physical Phenomena," etc., p. 7.

Maj. Gillette. 5.—"The presentation of an original form (in plan) of breakwater, whereby the natural energies are materially *aided,* without serious interference with either the flood or ebb forces."

This proposed form is certainly "original." Its principles are: First, by its cusps, to break up the oncoming tidal wave into opposing currents; second, to open and develop the beach channel, and, third, to protect the ebb channel from sands brought across the shoal by flood tides. Nothing like this breakwater in plan has ever been adopted by Professor Haupt or anyone else, nor has any breakwater covering the first or second principles been since built or proposed. Detached breakwaters between the littoral drift and the channel have been proposed, which would, incidentally, keep out of the channel a part of the sand brought over the shoal "by the flood tide," which sand, however, would be a small part of that driven along the coast by the waves and breakers. So far as known to the writer, no such windward detached breakwater has been built since the date of the above paper, the curved north jetty at Aransas Pass being, as will be shown, on the opposite side of the channel from the direction from which the resultant littoral drift comes. A windward jetty was built at this same place in 1881 to 1885 by Colonel Mansfield. A windward single jetty may have some good points—a windward detached breakwater has fatal defects.

6.—"A method of improvement whereby the *internal* currents are concentrated and conserved for more efficient scour after passing the gorge."

The principle of the paper has not heretofore been discussed, since, as a general proposition, internal works to cause any manifest difference in the improvement of the outer bar would have to be very extensive and of proportionately large cost. So far as known to the writer, none of the internal works suggested in the above paper have since been built or proposed.

7.—"A plan for utilizing the natural tendencies of the flood to cut a beach channel which shall be available for the lighter draught vessels."

As indicated above, this proposition has many grave objections. It would open the way for nearly all the littoral drift to be carried directly into the harbor; it would allow the escape of a percentage of the ebb tide, lessening the amount available for scour of the bar, which, in many harbors, would be a serious objection. It has never been used or proposed by anyone else.

8.—"The enunciation of the principle that the cause of the angular movement of the ebb stream after egress is due to the general form of the exterior coast line, which causes a racing of the tidal crests, from the outer capes toward the bight of the bay, and

that the *flood components thus generated are the principal forces* Maj. Gillette.
which build the bars and shift the inlets. This incessant semi-diurnal action of the flood is the *controlling element* in the forces affecting the magnitude and position of the bar. Storms and winds may modify and shift the deposits, but eventually the flood re-establishes the original conditions."

This paragraph is the gist of the paper. Its accuracy has been discussed above. Further comment is unnecessary.

9.—"The free circulation and ingress given to the flood by the detached breakwater, so designed as both to oppose a portion of the flood and produce interfering waves which deposit sand outside of the channel while it also aids the ebb in its attack on the bar by defending its channel and concentrating its volume."

The advantages of "circulation" are not patent, but the freedom of flood ingress to a harbor is a necessity. The scheme provides for this, but, as shown above, it provides for nothing else that has a practical value.

In one of his earlier projects, when the cost of high-tide twin jetties was too great for the existing commerce, General Gillmore provided for low-tide jetties, with a view to admitting the flood tide freely over their tops and directing the scour of the ebb tide between them—a logical plan having some advantages. Ever since that date the danger of not admitting the flood freely has been frequently urged against all twin-jetty constructions, but, in addition to what is said in the writer's paper, if such jetties are properly located there appears to be no real reason why the flood cannot get in through the same space as that through which the ebb escapes. Assuming high-water jetties, with their ends a reasonable distance apart, there is every reason to believe that the cross-section between them will be scoured enough to let the ebb fully escape; and, as the ebb has a volume always as great or greater than the flood, the latter should have no difficulty in getting through the same space.

10.—"For a given site and stage of water the flood movement approaches in the *same direction,* hence the resisting and regulating works should be placed on the near side of the proposed channel. If on the far side, they would be worse than useless, unless for shore protection."

The principle laid down in this paragraph is unquestionably wrong. So far as known to the writer, at every harbor, opening upon an approximately straight coast, where the matter has been tested, the direction of the approach of the flood is always modified by the wind, and often depends entirely upon it.

As shown above, and as now generally accepted, it is not the direction of tidal approach, but the direction of the littoral sand drift that is the controlling feature in jetty designing.

Maj. Gillette. 11.—"No artificial reopening of an outlet which has been closed by the flood component can be maintained without auxiliary works to deflect and modify its action. Dredging is only justified when the interests of navigation are sufficient to maintain a continuance of the expense and no other reasonable methods are available."

If "littoral drift" be substituted for "flood component," this paragraph will state a fact of minor importance, since it can only affect insignificant harbors.

12.—"The ability resulting from these general principles to construct works requiring a lesser linear development which will produce greater navigable depths at less cost."

It has been demonstrated above that the claim of this paragraph is fallacious.

13.—"The abolition of the risks and difficulties attending the navigation of narrow jetty entrances in time of danger."

This claim falls with the others. The ends of twin jetties are easily marked, and, especially if raised above high-water mark, make a very practicable entrance.

14.—"It frequently happens that the requirements of navigation and tidal concentration are conflicting, the former demanding wide entrances, the latter, on account of insufficient tidal volume, narrow ones. This debars the usual jetties and prevents improvement. The plans herein proposed are eminently adapted to meet such exigencies. As, for example, at Absecon Inlet."

This applies only to insignificant harbors, and wide entrances of any kind are usually impracticable at small harbors.

Above are shown the principal inaccuracies of Professor Haupt's paper, for which he received the prize from the American Philosophical Society and upon which he claims to base his present designs.

Its shortcomings may be briefly stated as follows: It takes no account whatever of the dry sand blown along shore and into the harbors and their entrances by the wind, which, in many harbors, is an important matter. It takes no cognizance of the cyclical movement of the channel across the bar, which is the most important element in the problem. It recognizes the fact that the channels were frequently "flexed" in the direction of the littoral drift, which flexure the paper attributes to the wrong cause, but overlooks the fact that they are frequently flexed temporarily in the other direction, in which position the channel is usually deepest, so that the paper alone would lead to the construction of many erroneously located works. The paper not only neglects, but the author subsequently combats, the now generally accepted fact that the waves produced by local or distant winds are the all-important elements in

littoral drift. It omits from consideration the action of great ocean currents and eddies from them, which, in the principal locality discussed, *viz.*, the Great Southern Bay of the Atlantic Coast, are doubtless of much more influence than the locus of the "cotidal lines." Maj. Gillette.

JETTIES AT ARANSAS PASS, TEXAS.

In 1895 a private company, under the advice of Professor Haupt, Mr. H. C. Ripley, and Mr. George Y. Wisner, constructed a "reaction breakwater" for the improvement of the bar at Aransas Pass, Texas. It has been claimed that this structure is an exemplar of the principles laid down in Professor Haupt's paper, "Physical Phenomena of Harbor Entrances," above discussed. Let us consider this.

Its form was that of the letter, "*S*," with gentle curves. It was located on the north of the entrance. The following letter, signed by two of the consulting engineers, gives the principles upon which it is based.

Joint letter of Lewis M. Haupt and H. C. Ripley, Consulting Engineers, dated Philadelphia, June, 12th. 1895:

"Gentlemen:

"In accordance with your request, through Brewster Cameron, Esq., we have the honor to submit the following brief description of the proposed breakwater for Aransas Pass, Texas, and the results we confidently predict from its construction.

"The work will be entirely of stone, or of stone with a thin brush mattress extending under a portion of a *(sic)* whole of its length, which will be 6 200 feet. In section it will have a top width of 10 feet, rising to a height of 3 feet above the plane of mean low water. The slopes will be 1½ horizontal to 1 vertical, and the base will vary, according to depths, from 40 to 70 feet.

"In plan it will differ from the usual form of jetty or breakwater, being detached from the shore and located on the bar to the 'windward' of the channel. Its axis will be curved (compound and reverse) to produce reactions similar to those found in the concavities of streams, and having radii sufficient to maintain channels of the requisite depths, as revealed by existing curves and their resulting depths of over 30 feet, now found inside the bar. It is designed to fulfill the fundamental conditions of (*a*) arresting the littoral drift: (*b*) admitting the full tidal prism to the interior lagoons; (*c*) controlling the ebb currents and producing a reaction across the bar; (*d*) changing the conditions of equilibrium of flood and ebb currents in favor of the latter, and (*e*) of affording aids to navigation by a structure of only half the length of the usual convergent or parallel jetties in pairs.

"The work will be executed in two parts. The first will consist of 1 250 feet of completed breakwater and 2 500 feet of foundation extension; the second of 5 950 feet of completed breakwater and 250 feet of foundation extension. It is to be covered with a substantial

Maj. Gillette. apron of heavy blocks, weighing from two to five tons, carefully placed so as to produce a permanent and substantial structure.

"The construction of the proposed breakwater, as designed, will unquestionably result in securing navigable depths over the bar of 15 feet for the first part of the work and 20 feet for the second.

"The development of these depths will commence immediately upon the construction of the foundation course, which will be greatly hastened by the strong currents resulting from the Northers, and which occur between September and March. If a more rapid development of depths is desired than will result from natural causes, deepening may be facilitated by dredging or other auxiliary appliances.

"Since the forces necessary to maintain a channel are much less than those required to create it, a channel once developed and protected from silt, as this will be, may be maintained by the natural tidal currents from the bays; hence the cost of maintenance under the plan proposed would be a minimum.

"Respectfully submitted,

"Lewis M. Haupt,

"H. C. Ripley."

By comparing these principles with those laid down in "Physical Phenomena of Harbor Entrances," and discussed above, it is seen that the structure is radically different.

1.—It hasn't any cusps directed against any "flood component" to break it up into currents which neutralize each other at the bottom of the "coves." In fact, neither the "flood components," "flood resultant," nor "cotidal lines," are mentioned or considered in its plan.

2.—It has no wing to improve the beach channel.

3.—It is claimed to be on the "windward" side of the channel, *i. e.,* the side from which the wind waves (not the flood tide), drive the sand. (As a matter of fact it is on the other, or "lee" side.)

4.—It is to be curved to produce "reactions" across the bar, similar to those found in the concavities of streams. Neither "3" nor "4" are mentioned in "Physical Phenomena of Harbor Entrances."*

Of course, the fundamental ideas of keeping sand out of the channel and increasing the eroding power of the ebb are common to the two ideas, as they must be to almost any system of structures for bar improvement.

We have here, therefore, as compared with the theories and plans of "Physical Phenomena," etc., an entirely new system of bar im-

* Curiously enough, the Aransas Breakwater has the same general shape as an internal "regulating deflector or reaction dyke," discussed in "Physical Phenomena, etc.," pp. 4 and 16, to prevent the interferences of "the confluent ebb streams of the inner basin" and make them "commingle and unite their energies" so that "the crossing on the bar" will "be consequently deepened." While this internal structure is called a "regulating deflector or reaction dyke" the reaction idea is wholly different from that at Aransas Pass, as the effect to be produced is not in the vicinity of the breakwater at all, but by training the ebb streams together to cause a deepening on the bar several miles away.

provement, differing completely from anything the writer can find in Professor Haupt's writings at all justifying his claim, above, that: Maj. Gillette.

"All which features have been frequently submitted to the Boards of Engineers in connection with proposals for the opening of ocean bars elsewhere with guarantees of depths since 1888, but which have not been made public by them nor reported to Congress."

Certainly, if Professor Haupt has nothing better than the ideas shown in "Physical Phenomena," etc., and "Dynamic Action," etc., and Boards have refrained from presenting them to Congress, the time of Congress has been mercifully saved.

But the new ideas involved in the "reaction breakwater," as exemplified at Aransas Pass, may be of more value. Let us consider it on its merits.

The general theory will be first discussed, and then the Aransas Pass example will be considered.

The fundamental principles which are to be applied in the "reaction" system, of which this is claimed to be an example, are:

1.—To arrest the littoral drift;

2.—To admit the full tidal prism to the interior lagoons;

3.—To control the ebb currents and produce "reaction" across the bar;

4.—To change the "conditions of equilibrium of flood and ebb currents in favor of the latter";

5.—To afford aids to navigation by a structure of only one-half the length of the usual convergent or parallel jetties in pairs.

Let us consider these in order:

1.—To arrest the littoral drift.—Assuming for the moment that the littoral drift is from the north, the "reaction breakwater" is supposed to be placed on the north of the entrance to act as a barrier by depositing this material on its northerly side. As this drift is mostly along the shore, it would undoubtedly be driven directly into the channel through the gap left between the breakwater and the shore. This is recognized by everyone at all familiar with the subject. Professor Haupt has frequently urged it as an objection to the low portion, near shore, in the north jetty at Charleston. Mr. Wisner, another of the designers of the Aransas Breakwater, also recognizes the fact.

He says:

"The action of the littoral current in flowing past any system of jetties is to pile up the sand drift in the angle of the shore and jetty on the windward side."*

If the structure is detached, the "piling up" will be through the gap and into the channel.

* *Transactions*, Am. Soc. C. E., Vol. XLII, p. 514

Maj. Gillette. This is a fatal defect in any bar structure, because the sand thus washed in goes into a part of the channel where it is bound to cause endless trouble. With twin jetties, or their equivalent, this sand would be scoured seaward and deposited on the bar—the very worst place for it.

Little or no sand drifts in through the gap at Aransas Pass, for the reason, as will be shown further on, that the drift is not from the north, a prominent element in demonstrating this being the fact that the breakwater has not caught one pound of sand on its northerly side, but, on the contrary, the sand there has scoured out since the breakwater was built to the amount of about 2 000 000 cu. yd.

If such a breakwater were the only structure provided for the improvement of the bar, without any natural or artificial structure

SOUTH WEST PASS, MISSISSIPPI RIVER.
IMPROVEMENT PROPOSED BY L.M. HAUPT.

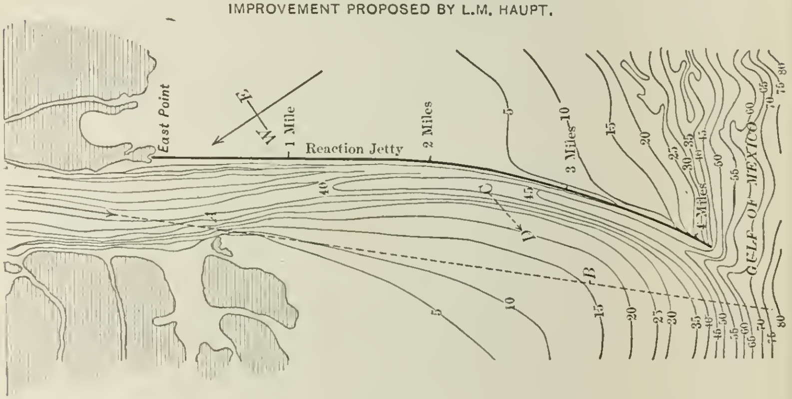

Fig. 37.

opposite to it to constitute a second jetty (as exists at Aransas Pass) and if it were located in accordance with the theory—on the "windward" side of an entrance—the littoral drift through the gap between the breakwater and the shore would work seaward between the breakwater and the channel, and surely drive the channel entirely away from the breakwater and leave it as the backbone of a useless, if not injurious, sand bank. Even without the gap, this second jetty would be necessary. This is illustrated by the diagram, Fig. 37, which is an exact copy from a paper, by Professor Haupt, on the "Reaction Breakwater as proposed for the opening of the Southwest Pass of the Mississippi River."

The diagram gives the condition that Professor Haupt expects would result from the construction of the curved jetty as shown. The arrow has been added to show the direction of the current as

it approaches the region of the jetty. This current would strike the bank near the point marked "A." Impinging thus against the soft mud at that point it cannot avoid eroding it, and there is nothing to prevent its ultimately leaving the proposed channel and going in some such direction as "*A-B*," without any reference whatever to the curved jetty. This is practically sure to happen in any case. Maj. Gillette.

2.—To admit the full tidal prism.—Assuming that the structure, in the general case, would admit the full tidal prism, it would also admit the full littoral drift, with the defects described above, and, as has been previously demonstrated in this paper, the admission of the full tidal prism is not difficult with any recognized method of bar improvement.

3.—To control the ebb currents and produce "reaction across the bar."—The structure will undoubtedly have effect upon the ebb currents if the ebb currents are forced to flow against the structure. This can only be done by something equivalent to a second jetty, since, in the general case, the littoral drift through the gap, above spoken of, must otherwise inevitably cause the channel to wear away from the breakwater to such a position that the latter would have no effect upon the ebb currents. In relation to this, it should be stated that although the system is claimed to require no second jetty, yet, as far as known to the writer, this detached breakwater has never been proposed by its designers for any entrance where there is not a *de facto* second jetty of some kind.

The sand coming in through the gap would form a shoal on the channel side of the inner end of the breakwater, which shoal, if there were no second jetty, would inevitably travel seaward along the face of the breakwater, and no force, except a structure opposite, has ever been suggested to hold the current up to its work and make it scour this sand away.

The reaction principle is based upon the idea of the concavities in streams. It is a well-known fact that the concavities of the bends of all sedimentary streams contain the deepest water, and, by analogy, the curved breakwater might be expected to produce the same results. But the analogy is not complete. A river has a second bank, which holds the current against the curve. The curve and the bar opposite are, in each case, constantly traveling down stream. The greatest depth in the river is usually close to the concave bank—almost too close for sheltered river navigation, and altogether too close for safe ocean-bar navigation. The cause of the action is generally accepted as due to the boring action of eddies, the axes of which are often vertical, and which are caused by the friction of the swiftly running water against the bank. As these eddies move down stream, any particular spot in the bottom is subjected to successive currents of rapidly changing direction. It is quite evident that erodable material under water, if exposed to a

Maj. Gillette. current flowing in one direction, becomes arranged somewhat like shingles on a roof, so that the current may cease to act upon them. If the current is suddenly reversed, or changed in direction, the bottom is torn up, and many of the particles are put in suspension and moved short or long distances. This is why the deep channel is found near the concave bank, but such action at a breakwater would not only be dangerous to the structure, from undermining, but would make a channel wholly impracticable to shipping.

The sand of an ocean bar thus put in suspension by such a reaction along a breakwater would apparently be carried to sea and discharged on the bar, the same as material carried by any kind of current, but it has been urged by the designers that the theory of the reaction breakwater provides for another method of transportation of the eroded material, due to a somewhat different phenomenon, also observed in the concavities of streams, *viz.*, that the rush of water against the concavity banks it up above the normal level and causes a down draft at the bank and across the river in a helical direction, carrying the eroded material to the sand-bar opposite. As to this theory, while it is probably often true as to the helical movement of the water, it is wholly inapplicable to a breakwater for an ocean-bar improvement, as a rush of water toward the jetty, sufficient to produce such an action, would be very dangerous to shipping, since it would take a very swift current toward the structure to produce the considerable head at the breakwater necessary to cause the strong helical action required to carry eroded material across a navigable channel of practicable width. This action may possibly occur in very narrow or crooked streams with rapid currents, but it is well known that the growth of sand bars occurs generally in the relatively still water below points where the current leaves the bank, and is due to the material brought down the river from above, which is deposited in the still water below the point, the bar growing down stream.

The experiments showing this helical movement, described in the Encyclopedia Britannica a few years before Professor Haupt's original paper, are shown only for a very small and crooked stream and for a complete bend of 180 degrees. The helical movement shown was also limited to the water only. It has never been demonstrated that particles of sand from a caving bank land on the growing bar opposite. They are much more likely to land on the bar below the next point on the same side of the stream. The particles of sand are not carried in suspension, but are picked up and dropped, or rolled along, time and again. In case the curvature is complete, and the length of the bend is several times the width of the stream, it is conceivable that particles eroded from the upper end might land on the opposite bank at the lower end, but no such conditions can be duplicated on an ocean bar. The curvature there must be

very slight, the curve itself only a small part of a complete bend, and short in comparison to the ebb-stream width. For example, the curved breakwater proposed by Professor Haupt for the mouth of the Columbia was in length only about one-half the width of the ebb stream between it and Cape Disappointment, and represented only about one-seventh of a complete bend of 180 degrees. Its curvature is only about 4 in. in 100 ft. Maj. Gillette.

Order of Work.—Great results are claimed for the peculiar system of construction involved in the theory of the reaction breakwater, which consists in building shoreward from the outer end,

"Whereby the scour is assisted by gravity and the advance of the bar seaward is prevented."*

"The reversal of the usual mode of construction by which the force of gravity is employed to assist in the erosion of the channel thus rolling the material down instead of up hill by building from the outer end shoreward."†

"From a depth exceeding 40 feet, the bed rises 14 feet in less than one mile, or one foot in 360, thence it flattens in the next two miles to one on 690, running nearly level for several thousand feet and suddenly dropping off with an outer slope of about one on 12 to the 25 foot contour. Thence it deepens rapidly to over 70 feet within a half mile.

"This profile is characteristic of a delta bar rolled seaward by a sedimentary river. All the material passed over it must first be dragged up the inner slope by friction or be carried in suspension by the ebb stream. With the bar removed the resistance to the transportation of sediment would be greatly reduced and if a lateral instead of a longitudinal movement be imparted to it, the path will also be shortened and the work required of the current be still further diminished. In short, the bar would be rolled over laterally instead of being pushed out into the Gulf in front of the jetties."‡

The inner slope up which the material has to be dragged has a maximum grade of 1 in 360. This is wholly negligible in the problem under consideration. On the other hand, under the supposed conditions after improvement, a quantity of material arriving at "*C*" (Fig. 37), to be rolled sideways, would have to go on about the line, "*C D*," which, from the 40-ft. contour up to the 20 ft. contour, has an average slope of 1 in 125 and a maximum much steeper. Considering the fact that the cross-current can hardly be as strong as the direct ebb current, it is difficult to see how "the work required of the current" will thereby "be still further diminished." This part of the theory, therefore, does not appear very reasonable. It has never been given a practical test.

The advantage of rolling the material down hill on the exterior

* Professor Haupt, in *Transactions*, Am. Soc. C. E., Vol. XLII, p. 499.

† Appendix to proposal of Reaction Jetty Co. for improvement of Columbia River Bar.

‡ Prof. Haupt, in a paper published in the *Journal* of the Franklin Institute, 1899, on S. W. Pass, Mississippi River, p. 5.

Maj. Gillette. slope is rather dissipated by the fact that such "hills" are of very gentle grade. For example, the exterior slope on the Columbia River Bar, parallel to the proposed "reaction jetty" is only 1 in 230, much more gentle than many of the grades on the Pennsylvania Railroad, and of trifling advantage for rolling sand on a sand bottom.

4.—Changing the condition of equilibrium of flood and ebb currents in favor of the latter.—Whatever meaning may be involved in this expression has never been explained by Professor Haupt. It does not appear in that form in "Physical Phenomena of Harbor Entrances," nor is there any idea in that paper which apparently means the same thing. As far as the writer knows, it first appeared in a letter from Professor Hilgard to Professor Haupt, dated May 20th, 1888,* in the following shape:

"Your plan would change the conditions of equilibrium in favor of the ebb."

This verbiage has since been used frequently by Professor Haupt, but he has never explained exactly what it means. The following quotation, though, throws some light upon it, referring to the two-jetty plan:

"And that such construction confined the movements at all stages of both ebb and flood to the same path across the bar, thus producing no changes of equilibrium in favor of the ebb," etc.†

The idea of having several inlets for the flood and only one for the ebb may be a good one, theoretically, but, as shown above, all plans yet devised for carrying it into effect, including the one under discussion, have serious defects. Incidentally, it is to be noted that this form of changing equilibrium was not provided for very well in "Physical Phenomena"—as it was there proposed to improve two channels, both of which would be used by flood and ebb, possibly, however, to a slightly different degree.

5.—Affording aids to navigation by a structure of only one-half the length of the usual convergent or parallel jetties in pairs.—This would be true if the breakwater were successful and were used alone.

The foregoing covers the general theory of the reaction breakwater. The alleged application of it at Aransas Pass should now be considered.

The Reaction Breakwater at Aransas Pass, Texas.

To determine how much this structure is a test of the theory for the construction of such breakwaters, it is necessary first to determine whether or not it is on the "windward" side of the channel, and, second, to determine what causes, outside the theory, have contributed to the effect it has had on the bar. Figs. 38 to 46, inclusive, show the condition at various times between 1895 and 1904.

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* See "Dynamic Action of the Ocean in Building Bars," p. 20.
† Professor Haupt, in *Transactions*, Am. Soc. C. E., Vol. XLII, p. 498.

Maj. Gillette.

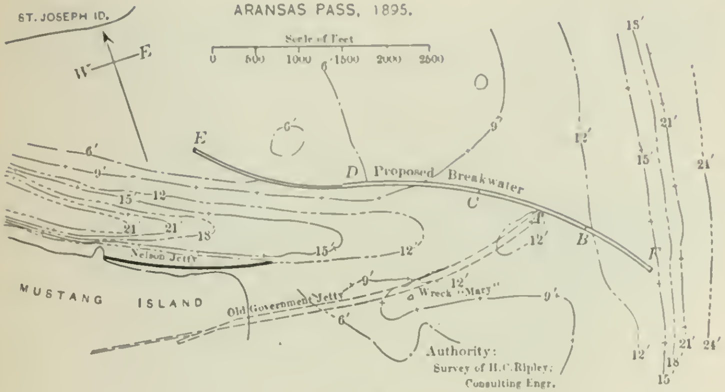


FIG. 38.

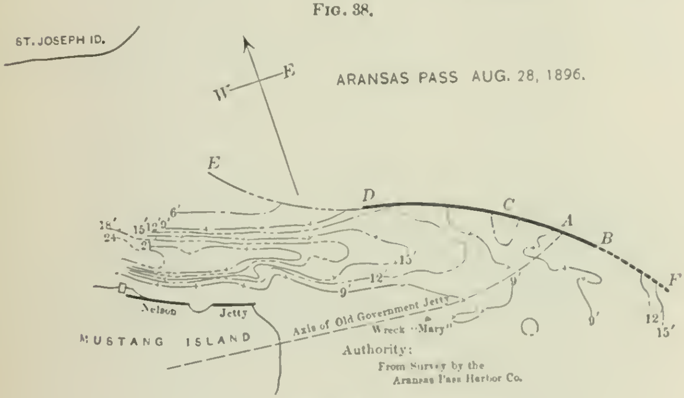


FIG 39.

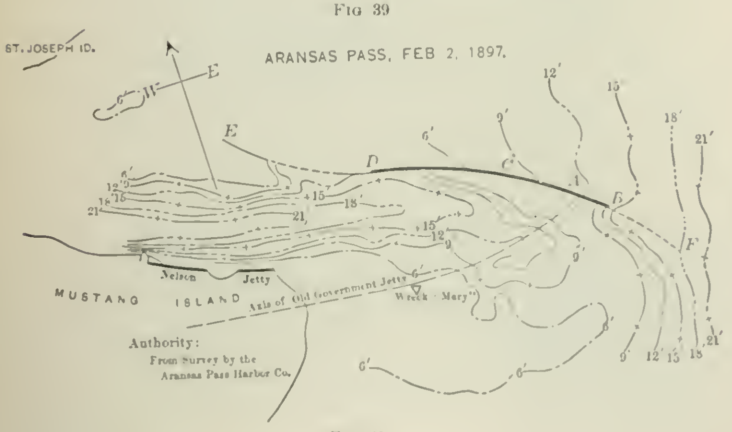
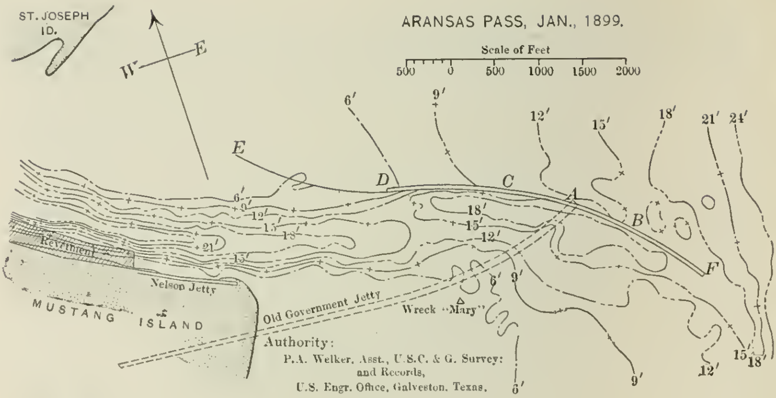
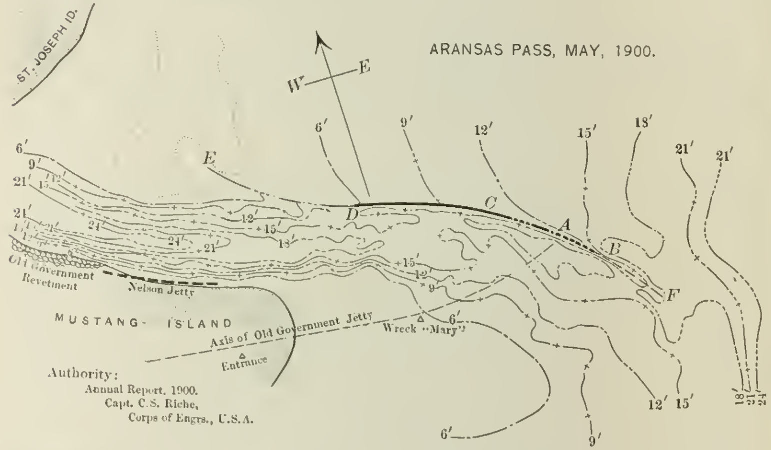
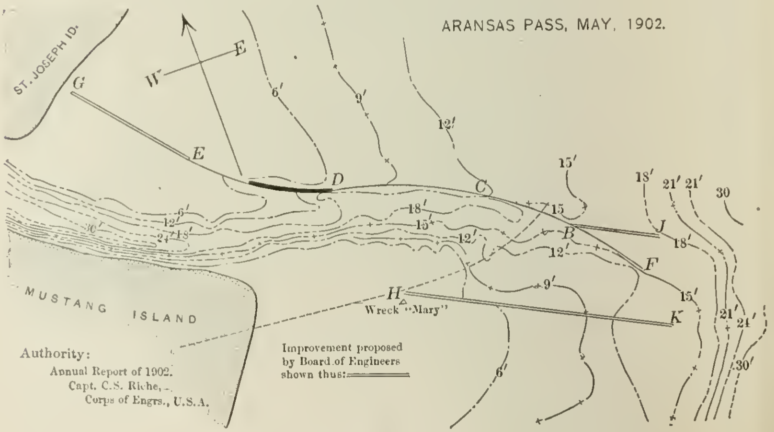


FIG. 40.

Maj. Gillette.







FIGS. 41, 42 AND 43.

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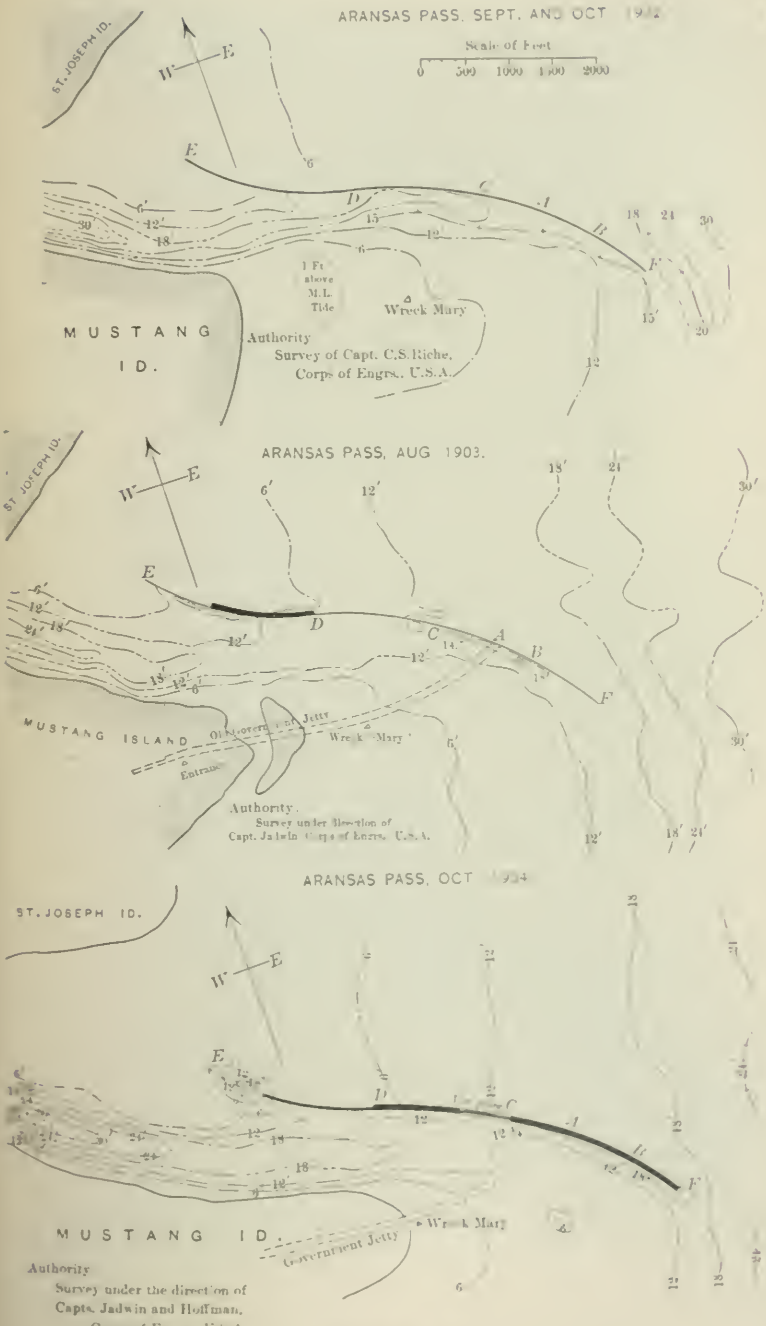


FIGS. 44, 45 AND 46.

Maj Gillette. As has been indicated heretofore, the writer believes, beyond all doubt, that this breakwater is on the "lee" side of the channel and not on the "windward" side, where it would intercept the littoral drift. The designers of the breakwater maintain the opposite, and it is important to determine the facts.

As demonstrated in the writer's paper, the following physical features are the most important ones known to determine the direction of the resultant sand drift at this place:

1.—Flexure of the channel with the drift;

2.—Configuration of the beach;

3.—Stream deflection;

4.—Movement of the entrance.

None of these is infallible, and a discussion of the forces producing littoral drift is also advisable. These are:

a—Wind movement of sand on shore,

b—Sand movement by ocean and tidal currents,

c—Sand movement by wind waves.

1.—Flexure of the Channel with the Sand Drift.—Unless complicated by peculiar conditions, this is the best natural feature to indicate the direction of the drift. Its action has been explained in the writer's paper. It should be noted that the channel never wears or changes gradually against the drift. The gradual change is always with the drift, and the jump to the new channel formed at the beginning of a cycle is always toward the origin of the drift. This cycle is always well marked in harbors of sufficient size and sufficiently free from other complications to give a definite, well-marked channel. In case the entire bar is shoal and the channel only a slight depression, it may sometimes be subject to fluctuations in all directions without any definite rule.

To determine the facts at Aransas Pass, the writer has had every available survey platted to the same scale and the channel lines indicated upon the same map, Fig. 47.

Previous to about 1881 the entrance moved south so fast, and the surveys made were so far apart, that it cannot be definitely determined whether the changes in the bar channel were due to a steady drift or sudden jumps in either direction. In only one series are there any consecutive moves in the same direction. These were 1868, 1871 and 1878, in which the channel appears perhaps to have been "flexed" to the south, and the channel of 1878, hugging the shore inside the wreck, "Mary," looks somewhat like a channel "flexed" by the littoral drift, but the channel of 1851 is almost equally positive the other way.

After 1881, owing to the Government work on the head of Mustang Island, the movement of the pass to the south entirely ceased. We find that there were two channels across the bar in 1882, the

Maj. Gillette.

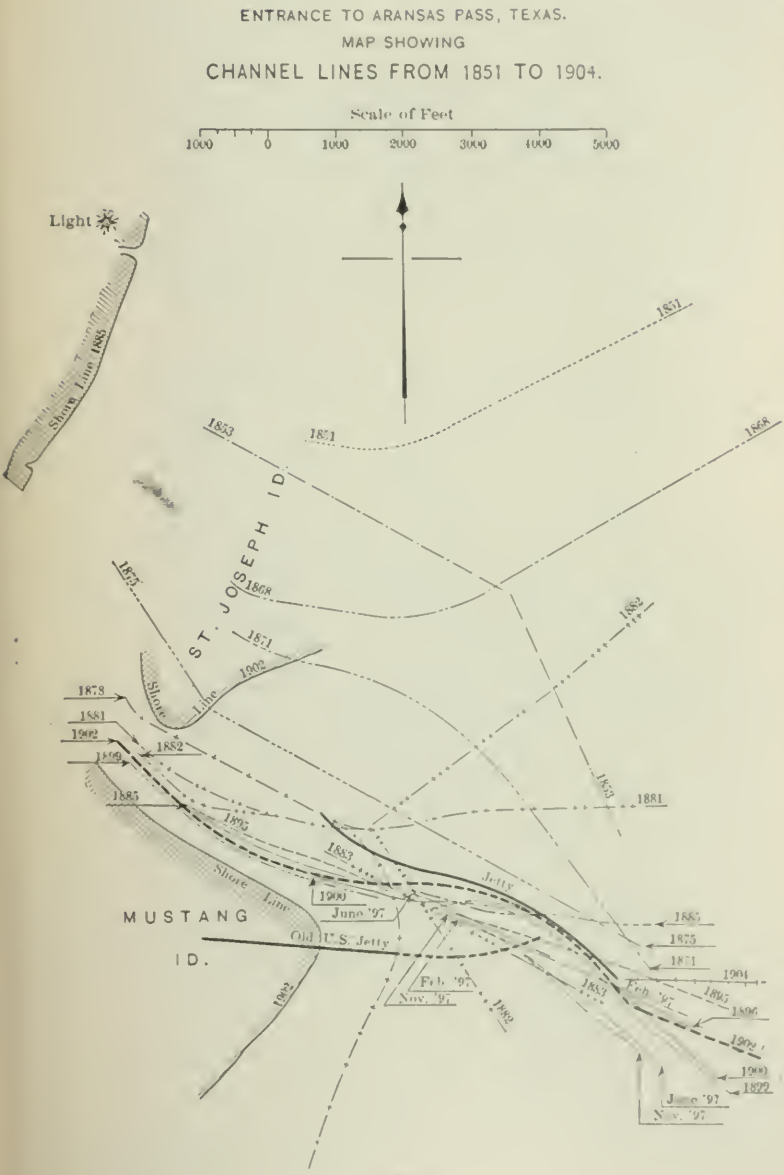


Fig. 47.

Maj. Gillette. channel of the previous year lying between them. The north channel was not as good as the channel of 1881, and continued to deteriorate and finally disappeared. The south channel of 1882 moved to the north in 1883 and again further to the north in 1885. This appears to be a complete cycle, indicating a sand drift from the south, but the construction of the Mansfield Jetty, between 1881 and 1885, was probably largely responsible for it. The movements of the channel between 1885 and 1895 are not known, as no maps are available, and since 1895 the conditions were changed by the construction of the breakwater. The short channel seaward of the breakwater has moved steadily north from 1900 to 1901, 1902 and 1904—a moderate movement, yet completely in accord with a littoral drift from the south.

The best that can be said of "channel flexure" at this point, as far as the maps are concerned, is that as yet there have been no complete records during a period when it could be considered untrammeled, except during the last four or five years, and then only for the extreme outer end of the channel. As far as this goes, the indication is strongly for a resultant drift from the south.

In the absence of a complete set of maps, testimony of competent persons on the ground must be considered. Mr. G. L. Webb, Assistant Engineer during the construction of the Mansfield Jetty, who appears to have been familiar with the locality for a longer period than any other competent person known to the writer, says:

"For the greater part of the past year the current has been running toward the north, the effect of the prevailing south winds. One effect of this current and the winds that caused it is to constantly move the bar channel toward the north, both by the direct effect of the wind on the pass current and by the pouring in of the Gulf water over the south bank, which causes the north bank to be cut away and the south bank to build up by the deposit from the Gulf water. During this action the bar gets longer, more particularly from the deposits on the in-shore side, and the channel becomes less defined. This action continues through one or more seasons until some unusually strong ebb current cuts a new channel to the southward, which soon becomes the main channel; the northeast channel closes up and the new channel begins moving toward the north, to be in turn closed up and replaced by one further south."*

The writer can find no competent testimony indicating a flexure in the opposition direction.

Taken altogether, the "flexure of bar channel" test indicates a littoral drift from the south.

2 and 3.—Configuration of the Beach and Stream Deflection.— There appears to be no record of these available, but the offset of the

* See Annual Report of the Chief of Engineers, 1882, p. 1474.

Maj. Gillette.

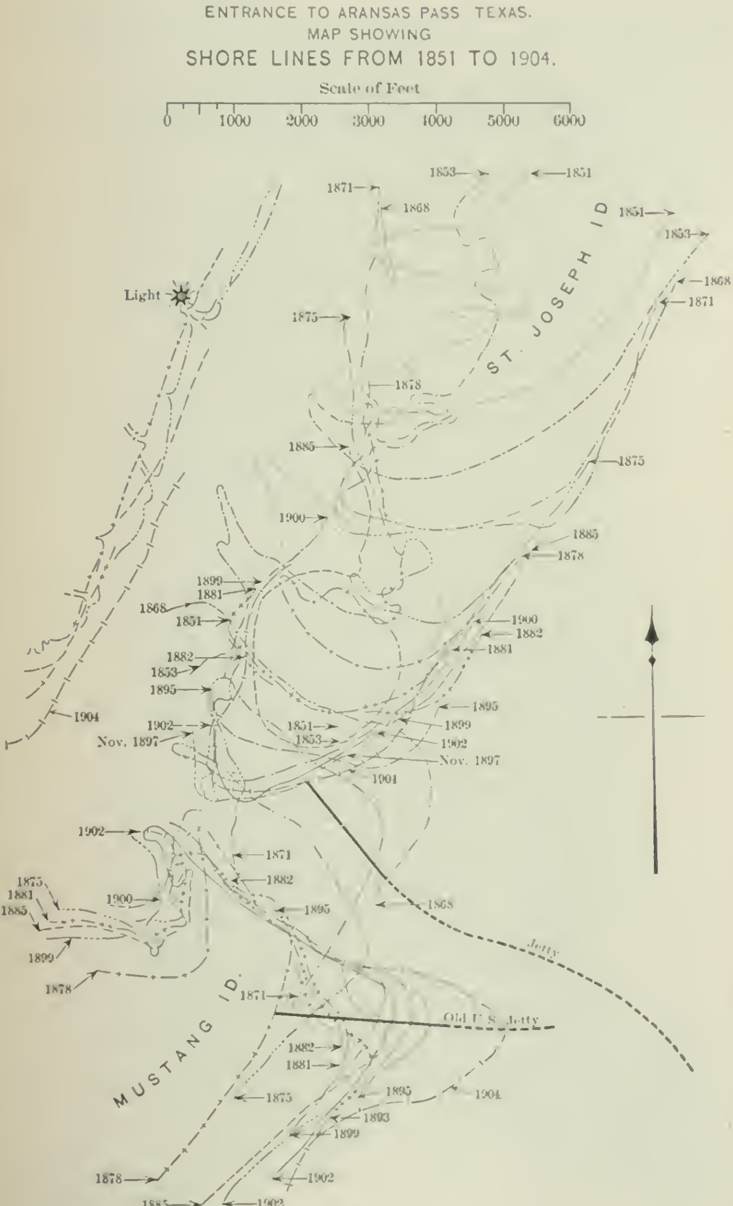


FIG. 48.

Maj. Gillette. south side of the entrance indicates a drift from the south. The force of this, however, if lessened by the method of movement of the entrance, is discussed in the next paragraph.

4.—Movement of the Entrance.—In many cases inlets travel with the littoral drift, but the cases given above, where the opposite is true, make it by no means a sure criterion, as Professor Haupt would have it.

In the western part of the Gulf, all the inlets move to the south. The reason of this is about as follows: The shallow lagoons inside of the sand cordon, built up by the waves of the Gulf, have habitually only about 1 ft. of tide once a day. At certain seasons, however, it is a frequent occurrence for a southerly wind to blow for a time until the lagoons or bays are raised above their normal level, then the wind changes almost instantly to a strong wind from the north. These are known as "Texas Northers." They start at full speed, and the swollen waters in the lagoons are driven violently to the south, escaping to the ocean with great velocity through any inlets near the southern part of the bay, scouring a deep channel and rapidly eroding the south side of the inlet. This, then, is the cause of the movement.

If this movement were caused by the littoral drift the island north of the inlet would grow to the south and narrow the channel, forcing it to cut away its southerly side. This does not occur. On the contrary, the comparative map, Fig. 48, shows that the south bank cuts away and the north slowly follows it. From 1851 to 1853 the northern end of Mustang Island was eroded to the southward 1 100 ft., while St. Joseph Island followed only 900 ft. of this, showing that the channel of this interval was not pushed by any littoral drift from the north. From 1853 to 1868 Mustang Island moved south 1 400 ft.; St. Joseph only followed 700 ft. From 1868 to 1871 Mustang Island moved south 600 ft.; St. Joseph Island followed only 300 ft. From 1871 to 1878 Mustang Island moved south 2 100 ft.; St. Joseph Island followed 2 000 ft. In the aggregate, from 1851 to 1878, Mustang Island moved south 5 200 ft.; St. Joseph Island followed only 3 900 ft.; widening the inlet at its narrowest point, from 1 200 ft., in 1851, to 3 200 ft., in 1878.

This shows conclusively that for this period the movement of the pass to the southward was not caused by a littoral drift from the north. The abnormally wide opening in 1878 was subsequently contracted between that year and 1881 about 650 ft. by St. Joseph Island moving south 800 ft., while Mustang Island only moved south 150 ft. It is to be noted that in May, 1880, the revetment of Mustang Island was begun, so that a portion of this gain in movement by St. Joseph Island is accounted for by the stopping of the travel of Mustang Island. Since that date the head of Mustang

Island has been held in the same position while St. Joseph Island has moved south 2 050 ft., making the width now 850 ft., somewhat less than in 1851. Maj. Gillette.

The movement of this inlet southward, therefore, is no indication whatever of a resultant littoral drift in that direction. On the contrary, it is a strong proof against it. In fact, the sand supplied to the south end of St. Joseph Island in its delayed following of the inlet does not even call for any sand drift from the north at all. It is very simply accounted for as follows: All the charts show about the same bar opposite the entrance, whatever the position of the latter. The entrance has moved south 2 miles; the bar, apparently, has followed it, and up to 1881 there was at all times and in all positions about the same quantity of sand in the bar as there was 30 years before. In the meantime, part of the great quantity scoured off the head of Mustang Island must have gone out on the bar, which should naturally have greatly increased its volume. Therefore, the north side of the bar, or the bar of the earlier positions, has gone somewhere. The most natural place for it to go would be to the foot of St. Joseph Island, being simply driven inshore by the external forces of winds and waves. Previous to this it was kept out to its normal position by the ebb outflow. As the previous channel moved south, the ebb forces disappeared from the north end of the bar, and the external forces drove it in and built up St. Joseph Island.

The evidence of the inlet movement is then indefinite as to a littoral drift from the south, but quite positive against any large littoral drift from the north.

Let us now consider the known forces, and see what they indicate:

1.—*Wind Movement of Sand on Shore.*—Prevailing winds, up or down the coast, sometimes onshore and sometimes offshore, will move considerable quantities of sand before them. The direction of this movement will coincide with the direction of the movement by wind waves, and may be added to that movement wherever, as is generally the case, the prevailing winds are onshore.

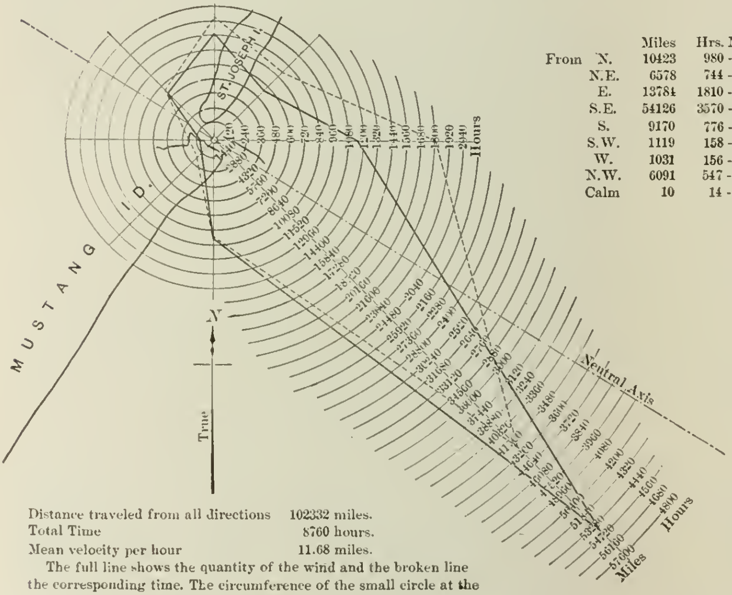
2.—*Sand Movement by Ocean and Tidal Currents.*—The only great ocean current in the Gulf is the Gulf Stream, but this portion of the Gulf is far removed from any indication of such a current. The points of entry and exit of the Gulf Stream being narrow channels, the wide expanse of the Gulf opposite Aransas Pass must practically dissipate the current and leave only a very gentle flow, not swift enough even to produce any eddies that would not be reversed by a very slight wind. The tides, too, are insignificant, so that, while the inflowing tide close to the entrance will doubtless carry in light material in suspension, it can have practically no

Maj. Gillette. influence upon the littoral drift. There being so small a tide, it is needless to say that the "flood component" and "flood resultant," as well as the position of the "cotidal lines" are negligible matters. Therefore, absolutely the only force left worthy of consideration as producing littoral drift at this point is the wind acting through the waves and currents which it produces.

3.—Sand Movement by Wind Waves.—Distant storms in the ocean would produce no waves affecting this part of the coast on account of the barricade of the West Indies. Local Gulf storms

DIAGRAM OF WIND AT CORPUS CHRISTI, TEXAS.

FOR ONE YEAR - DEC. 1901 TO NOV. 1902.



	Miles	Hrs. Mins.
From N.	10423	980 - 30
N.E.	6578	744 - 15
E.	13784	1810 - 20
S.E.	54126	3570 - 40
S.	9170	776 - 15
S.W.	1119	158 - 55
W.	1031	156 - 50
N.W.	6091	547 - 45
Calm	10	14 - 30

Distance traveled from all directions 102332 miles.
Total Time 8760 hours.
Mean velocity per hour 11.68 miles.

The full line shows the quantity of the wind and the broken line the corresponding time. The circumference of the small circle at the center is the zero of the scale.

Fig. 49.

and winds, therefore, are the dominating elements. The local winds at Corpus Christi, within a few miles of the Pass, must, in this level country, be identical with the winds of the Pass. These winds are very remarkable. A diagram of them, showing also the coast line, is given in Fig. 49.

This diagram is typical of every year, and shows a remarkable predominance, both in time and strength, of the winds from the southeast blowing at an angle of about 14° with the "neutral axis," a line at right angles to the general coast line. As these winds blew

during the year for 3 600 hours with an average velocity of 15 Maj. Gillette.
miles per hour, they must have produced a large movement of sand to the north. In this they were assisted by the south winds which blew for 775 hours at an average velocity of 11½ miles. The effective winds to produce movement down the coast were those from the east and the northeast, both insignificant in comparison with the southeast wind. Winds from the west, northwest and north are offshore winds, and are wholly ineffective to produce littoral drift. This southeast wind is the only conceivable force of any magnitude to produce littoral drift at this point, and it is simply impossible that it should have produced a resultant littoral drift from the north, but must have produced a great movement from the south.

It is to be noted that all the winds recorded as S. E. did not blow exactly from that point. They varied 10 or 12° on either side of the exact S. E. line. This would partly neutralize those that came from the more easterly direction, but more than make up for it by the much more effective angle of those more from the south. The drawing indicates the resultant.

Actual Measurement of Sand.—Additional evidence of this statement is furnished by the sand deposited against obstacles interposed in the way of the drift. Given a quantity of drifting sand under water moving in one direction, and an artificial obstruction from shore placed across its path, deposits, or scour, or both, may result, and it is a matter of some importance to determine what the result will be.

The theory of the designers of the reaction breakwater is that the drifting sand will be intercepted by, and be deposited outside, the breakwater, *i. e.*, on the side from which the drift comes, as the following quotations indicate:

"It is designed to fulfill the fundamental conditions of * * * arresting the littoral drift."*

"The conditions which this structure was designed to fulfill were:

"1. That it should arrest the littoral drift of sand and defend the channel from its encroachments—hence it must be placed to 'windward' of the proposed channel."†

"As a means to this end the breakwater * * * is also composed of curves whose radii and centers are adjusted to the site in such a manner as to cause deposits on the outer side of the structure."‡

The consensus of opinion agrees with this. Under ordinary conditions, a spur built out across a line of drifting sand will stop the sand before it passes the spur until it banks over it.

Three spurs, jetties or breakwaters, have been built at Aransas

* "Joint Letter of Messrs. Haupt and Ripley."

† Professor Haupt, in *Transactions*, Am. Soc. C. E., Vol. XLII, p. 192.

‡ Professor Haupt, in *Transactions*, Am. Soc. C. E., Vol. XLII, p. 199.

Maj. Gillette. Pass at different times, and their effects should be almost conclusive as to the direction of the littoral drift. In 1869 a spur 600 ft. long was built from St. Joseph Island. It was followed by a deepening of the bar channel some distance away, about 2 ft. There is no available evidence to indicate upon which side it stopped the sand, as it lasted only for a short time. Its improvement of the bar has been attributed to its checking the littoral sand drift from the north, but, as it was built to "close a secondary channel,"* its effect on the bar, if it had any, is more readily accounted for by the closing of the secondary channel than by assuming a littoral drift from the north.

The second was the Government Jetty, built on the south side of the channel, between 1881 and 1885. This was built mostly of brush mattresses which were then being tried on the Gulf Coast. They proved unsuccessful, as they settled, were torn out by waves, and were destroyed by the teredo, so that the jetty only remained to its full height for a short time. While it stood up, however, there was deposited on its southern side about 65 000 cu. yd. of sand. At the same time there was scoured out from the area inside the jetty on its northern side about 166 000 cu. yd. of sand. As the jetty was lowered, both of these effects were lessened. A comparison between this jetty and the "reaction breakwater" will be given further on.

The third jetty was the "reaction breakwater," built in 1895-96 on the north side of the channel. This breakwater, being designed to catch and store on its northern side the drifting sand from the north, should have done so, at least throughout its extent and to its height, had the littoral drift been from that direction, but it absolutely failed to do so. Instead, there has been, since its construction in 1895, a scour of about 2 000 000 cu. yd. of sand from the shoals north of it. This alone is almost conclusive proof that the littoral drift is not from the north, but from the south. At the same time, there has been a scour in the channel south of this breakwater. This scour is accounted for further on as the action of twin jetties. Immediately south of this there has been a fill of 1 334 000 cu. yd.

In this connection, Professor Haupt does not appear to have clear ideas always as to where he intended the breakwater to stop the drift, whether on its northerly side toward the alleged source, as he stated previously, or on the southerly side to be scoured out by the currents, as the following quotations will show.

Speaking of the Aransas Pass breakwater, he says:

"The partially controlled currents have actually removed about 400 000 cu. yd. of compact sand from the bar without the aid of a dredge or other mechanical appliance, and have prevented the depo-

* "Dynamic Action, etc.," p. 21.

sition in the channel of a much larger volume driven along the coast by the angular wave movements."* Mr. Gillette.

This statement was criticised as follows:

"This is correct as far as the amount removed from the channel is concerned, the cause of which removal will be discussed further on. But the statement that the work has 'prevented the deposition in the channel of a much larger volume driven along the coast by the angular wave movements,' and that the work is 'adjusted to the side in such a manner as to cause deposits on the outer side of the structure,' is wholly at variance with the fact that, on the contrary, as shown above, an enormous scour, over a million and a quarter yards, had taken place on the outer side of the structure.' This fact was fully indicated by the Coast Survey map referred to by Professor Haupt in the above quotation."†

To this criticism, Professor Haupt replied, as follows:

"Unfortunately for this argument, the original compact material in place * * * (*i. e.*, in the channel) has been scoured out by the natural currents even to a depth of over twenty feet and close to the breakwater * * * consequently any loose sand carried over the breakwater would, *a fortiori*, be much less apt to be lodged in these currents and would be at once carried out and around the sandy spur to the southward, as has happened and as is quite evident from the comparative charts; so that the statement by the writer is true that not only has this incomplete breakwater removed about 600 000 cubic yards in place, but has prevented the deposition of a much larger amount drifting in from the north through the gaps and over the unfinished portions of the structure. This action is so manifest as scarcely to require so long an explanation, but for the misconstruction which has been put upon it."‡

This would make it appear that the breakwater "prevented the deposition of the material in the channel" by permitting it, to the amount of 1 250 000 cu. yd., to go over the breakwater into the channel to be afterward scoured out. This is not at all in accord with the theory of the breakwater. It should be stated that this sand could not have done anything of the sort, else it would undoubtedly have filled up the north side of the breakwater to its crest or to the bottom of the gaps, and there was absolutely no fill in the vicinity at all, but a decided scour, the breakwater standing at all times at not less than about 10 ft. above the sand at its back.

Up to September, 1904, the total scour north of the breakwater since the date of its commencement was 1 956 500 cu. yd.

The only evidences produced by Professor Haupt, to support his claim of a sand drift from the north, are the movement of the entrance and a photograph of some vegetation.

* *Transactions*, Am. Soc. C. E., Vol. XLII, p. 499.

† House Ex. Doc. No. 35, 56th Congress, 2d Session, p. 28.

‡ *Proceedings*, Amer. Phil. Soc., Vol. XL, 1901, p. 13.

Maj. Gillette. The writer has not seen the photograph, but on his first visit to Aransas Pass, the shape of the sand dunes and the growth of the vegetation on them were the first striking indications that, as far as wind waves were concerned, the sand drift must be from the south. The writer has explored the head of Mustang Island thoroughly, and, while in places partly sheltered by other dunes indications of wind from two or three directions can be found, the great majority of the dunes indicate unquestionably a drift from the south.

The above covers every conceivable factor that can help determine the littoral drift, and it seems to the writer to be a demonstration. It shows the danger of trusting to one or two indications, for the designers of the breakwater, after producing much literature to show where others had "built the wrong jetty first," have, beyond doubt, fallen into the same error themselves.

Professor Haupt criticises the slight difference between the writer's opinion of the quantity of drift in 1901 and now. The recent view is the result of more complete investigation, and the facts shown by recent surveys.

Under these circumstances, the claim that the structure at Aransas Pass is an example of the "reaction breakwater" theory falls to the ground.

Work Accomplished by the Breakwater.—There remains to be considered the reason for the effect that the breakwater has had upon the channel across the bar.

From a careful study of the history of the work, and of the numerous surveys of the locality that have been made, with two examinations of the site, the writer's conclusion is that the breakwater, in connection with the other works at this place, acts simply as one of an imperfect pair of twin jetties. These two *de facto* jetties are shown in longitudinal section on Fig. 50.

Starting from a line about at right angles to the general direction of the channel and passing through the point where the breakwater, prolonged, would strike the shore of St. Joseph Island, we have a north jetty composed; first, of a wide natural bank, submerged a few feet, extending from St. Joseph Island to the inner end of the breakwater, a distance of about 1 850 ft., the direction of the ebb discharge being such that no artificial structure has yet been necessary to maintain this; next, we have, for a further distance east of 5 500 ft., in the breakwater itself, a nearly complete actual jetty.

Opposing this, and forming the south jetty, we have; first, the revetted head of Mustang Island, a substantial work of the utmost importance, every part of which appears to be in place; then, the old Nelson Jetty, the two extending from the same line for a distance of about 3 150 ft. Beyond that, we have the old Government Jetty,

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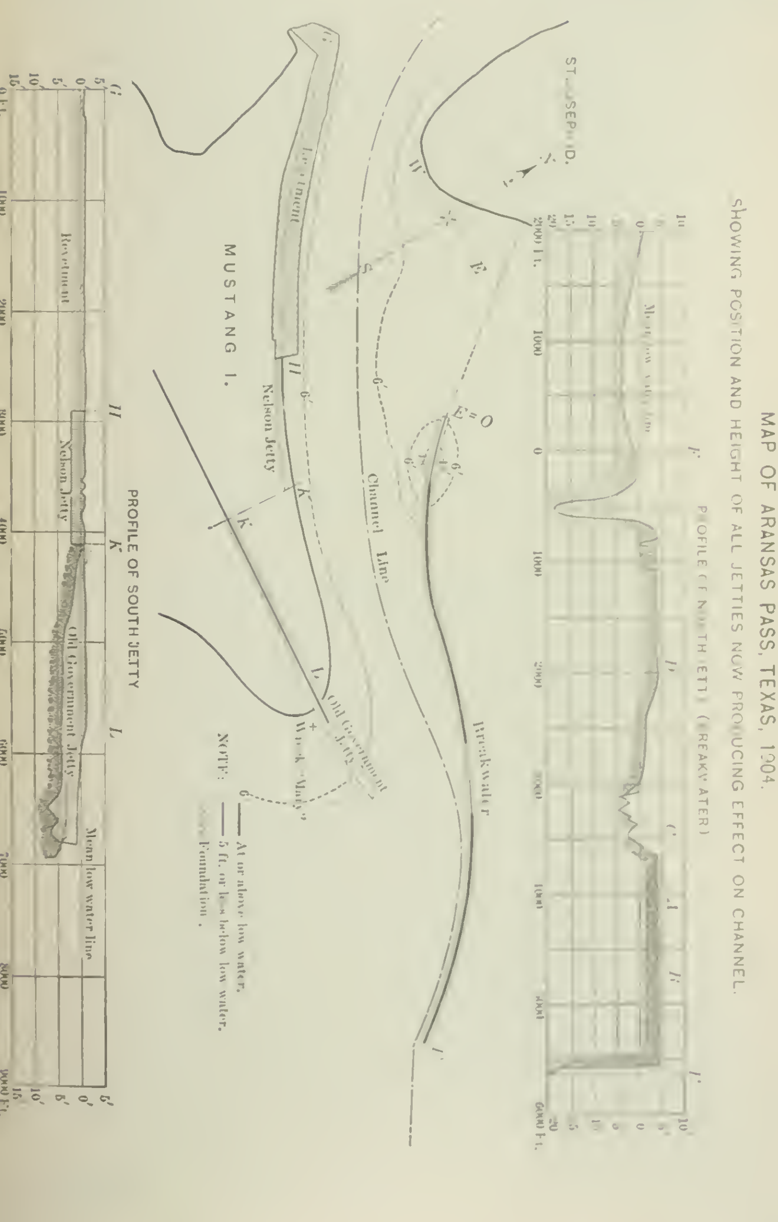

FIG. 60.

Maj. Gillette. a submerged structure, but still a jetty capable of exercising an important influence on the tidal flow, a further distance of 3 000 ft.

These make a north jetty, having a total length of 6 850 ft., and a south jetty, having a total length of 6 150 ft., located about 1 250 ft. apart, this distance narrowing with the increasing depth on the old Government Jetty toward its outer end. This old Government Jetty, although much wasted in its outer portions, has always had and still has an efficient action.

It holds the principal flow near the "reaction breakwater," which constitutes the north jetty, and prevents the formation of a deep channel anywhere except near the breakwater. Where a bar current is free to change its channel, a structure may cause the change and then appear to become useless by being buried in the sand. The old Government Jetty, from its shore end to some distance beyond the wreck, "Mary," undoubtedly has had this effect, although a portion of it is now underground. This fact is recognized, and rather over-valued, by Professor Haupt, speaking of the danger of undermining the "reaction breakwater":

"And with the submerged Government jetty reflecting the currents against its face it may well excite surprise that it has not been undermined long ago."*

Professor Haupt charges the writer with inconsistency in counting this old submerged work a jetty while calling the similar one at Columbia River "not properly a jetty at all." Professor Haupt did not get the point of the latter remark. The action of the two structures is quite similar, but the results in the latter case were simply to bring the "retreating south side of the Columbia out to the general line of the coast" ready for the commencement of a pair of jetties. In that sense, it was not a jetty at all.

It will be noted that the effective part of the north jetty is opposite the ineffective part of the south jetty, and *vice versa*. The defects of both these jetties are overcome by directing the current against the effective part of each in turn. Thus, the ebb current issuing from the bay is first directed sharply against the revetment on the head of Mustang Island, thence it is deflected against the breakwater, thus reducing the importance of the lack of strength and height in the natural submerged bank forming the inner end of the north jetty and of the lack of height of the old Government Jetty now forming the outer part of the south jetty. The latter is also helped at present by the sand bank drifting up from the south.

Thus far, the "reaction breakwater" at Aransas Pass has cost about $590 000. The works opposite to it, and which have materially aided it in the work it has done so far, have cost as follows:

Revetment of head of Mustang Island......$202 000.

* *Transactions*, Am. Soc. C. E., Vol. XLII, p. 543

Without this excellent and strong piece of work the entrance Maj Gillette would have long since traveled to the south and left the breakwater out of the problem.

The Nelson Jetty cost about................$108 000.

A part of it was still intact and visible above the sand at the time of the writer's examinations, and appears to help hold in position the shore line opposite the jetty, partly through its outer end acting as a groyne. Even if this were not so, it is backed throughout its full extent by the shore end of the Mansfield Jetty, which is undoubtedly practically in the same condition as when placed. This old Government Jetty cost about $350 000.

Without these three structures, the channel might long since have left the breakwater entirely, and would certainly have done so if the sand drift were from the north.

The aggregate cost of the above work, constituting the second jetty at Aransas Pass, is $664 500, so that, at this small entrance, the "reaction breakwater" is supplemented by a second jetty costing more than the breakwater. The latter is not yet complete.

These facts make curious reading of Professor Haupt's statement in his discussion (p. 346), that:

"There is to-day no other structure at that inlet which can be said to form a second jetty, and there has not been since this work was started in 1895."

These things must be taken into consideration in estimating the cost of a "reaction breakwater" as applied to places where there is no *de facto* second jetty.

To show that there is efficient twin-jetty action, imagine a current of tidal water flowing seaward just north of the Nelson Jetty. This water, unopposed, would tend to spread freely over a fan-shaped area. It is prevented from doing this by the breakwater on the north, and the jetty-ribbed sand bank on the south. This is concentration, pure and simple.

The outer end of the breakwater acts as a single lee jetty, and has the appropriate inherent defects. The sands, driven up the coast by the littoral drift meeting the effluent current, which is directed somewhat against them, have been and are building a sand bar opposite this part of the jetty, which is apparently being driven closer and closer to the breakwater, and, unless a south jetty is soon provided, this bank will envelop the end of the jetty in such a way as to block the entrance. The "channel" now cuts close around the end of the breakwater, and, while the recent building up of this part of the structure may cause a stronger current against the drift, it appears to be only a question of time when the sand bar will form around the end of the jetty and mask the entrance too

Maj. Gillette. much for practicable navigation. The length of time to do this will probably depend upon the frequency of strong currents driven out of the bay by "Northers." Without frequent "Northers," too, it is probable that the sand coming up the coast will continue to hold the channel so close to the curved part of the breakwater at its outer end, that it will remain impossible to navigate. At present, though Professor Haupt claims more than 20 ft. depth, the bar pilot will not undertake to bring in a ship drawing more than 10 ft. (less, if the boat is more than 150 ft. long). The narrow trench, or channel, which is carefully shown by Professor Haupt in section, but not in plan (Fig. 24), is not used at all by such boats as make the harbor.

The above deposit of sand on the south side of the channel, accompanied by the pronounced extension seaward of the head of Mustang Island (Fig. 46), is attributed by Professor Haupt to the action of his curved breakwater, and shows his appreciation "of a lateral displacement of material by reaction from a concave directrix and the construction by natural agencies of a convex bank at a safe navigable distance therefrom."

While, to a casual observer, the existence of this bank south of the channel seems as easily accounted for by Professor Haupt's "lateral displacement" as by a sand drift from the south, it seems to the writer that the facts given above show that it is the latter and not the former. If any further facts are needed, the following will apply.

Compare Figs. 38 and 39, showing conditions at the beginning and close of the "first part" of the work. The channel is breaking through in Fig. 39, away over toward the wreck, "Mary," and not close to the breakwater where "concavity of stream" action and "lateral displacement commencing on the outer scarp" should have it.

Professor Haupt describes this as follows:

"The currents are moulding their own channel, with a thalweg about 500 ft. from the axis of the work and parallel thereto."*

The currents were truly "moulding their own channel, with a thalweg about 500 ft. away from the breakwater," but they were not doing it by any helical action due to the structure, for if they were, such action should first have shown itself next the breakwater, where "reaction" should be the strongest. On the contrary, it had to wait for the sand drift from the south to fill in the south side of the channel and drive it against the breakwater, which action is progressive and continuous. (Figs. 40, 41, 42, 43, 44, 45 and 46.)

Let us now consider "reaction" in connection with the action of the outer end of the breakwater.

* *Transactions*, Am. Soc. C. E., Vol. XLII, p. 532.

The writer's contention, in the Report on Brunswick Bar and in the present paper, was that one of the objections to a lee jetty was that the channel would at times be driven too close to the jetty. Also, in both papers, that the "reaction" effect of a curved structure, when effective, would be to dig a channel much too close for ocean-bar navigation. The latter was disputed by Professor Haupt, who maintained that the channel would be at a "safe navigable distance therefrom." Maj. Gillette

The recent work, finishing the breakwater from *"F"* nearly back to *"C,"* was begun in July, 1903, and finished in September, 1904 (Figs. 45 and 46). As there is no jetty opposite this part of the work, "reaction" should have been untrammeled. What has it accomplished?

First.—The thalweg is now right on the rocks at the toe of the breakwater. What drove it there? Is it not unquestionably the result of the sand drift from the south combined with the "reaction" of the breakwater directed toward its own undermining?

Second.—When the work was begun in 1903 the controlling depth was 14.7 ft., located on a shoal opposite a point about half way between *"A"* and *"C."* Now it is still at the same point, and the depth is 12.3 ft.

Third.—When work was begun the 12-ft. channel was about 400 ft. wide. Now it is about 100 ft. The trench along the toe of the breakwater is deeper than it was, and the sand seems possibly to have been "laterally displaced" a few yards. Its amount is obviously much less than the increase in the fill in that vicinity. Where did the latter come from? It cannot be accounted for except as a littoral drift from the south. And, whence came the sand that shoaled the controlling depth 2.4 ft.? "Reaction" could not have put it there, for all the "reaction" shown is further seaward. A scour occurred in the channel a little further inshore on the inner scarp, and the sand obviously came from there. Equally obviously, this scour was due to the narrowing of the channel by the sand drift from the south over the region of the old Government Jetty. "Reaction" could not possibly have caused it, because it is opposite a portion of the breakwater, convex to the channel.

The difference in the action of the "first part" of the work, Figs. 39 and 40, and the recent work, Figs. 45 and 46, is noticeable. In the former case twin-jetty action dominated "reaction," and the channel opened some distance from the breakwater, as was normal for twin jetties. In the second case, "reaction" was effective enough to start the structure to digging its own grave, in which rather beneficent work it was aided by the sand drift from the south.

The Old Government Jetty. Before leaving this subject, it seems pertinent to give the facts concerning the old Government

Maj. Gillette. Jetty, which has been many times referred to as an exemplar of wrong location and also as preventing the proper action of the "reaction breakwater."

The following quotations bear upon the subject:

"It merely intercepted the littoral drift moving southerly, dropped it in the channel which it obstructed and pushed the bar seaward with a consequent loss of depth. It cost nearly half a million dollars and was an acknowledged failure. It furnished a complete demonstration of the falsity of the theory of attempting to create a channel by placing a jetty to 'leeward' of the channel."*

"Then the old Government jetty built out about one mile into the Gulf and on that (the south) side of the entrance, prior to 1885, should have deepened the channel by intercepting the drift on its 'windward' side. Unfortunately for the argument, the sand gathered on its northern or leeward side and was dropped in the channel, thus blocking it up, and the bar advanced seaward as fast as the jetty was extended, so that the official reports stated the deepening to have been insignificant, and the work was abandoned to private parties."†

The writer is constrained to contradict flatly almost every statement in these quotations.

As shown above, this jetty, instead of merely "intercepting the littoral drift moving southerly," intercepted the littoral drift moving northerly, and kept 65 000 cu. yd. of it from going into the channel. Instead of obstructing the channel, it deepened it. It was built between 1881 and 1885, beginning with 6¼ ft. on the bar.‡ Being built of brush, a material which had been found successful abroad, but which failed here, it was shortly afterward so reduced in height as to be inefficient, but while it lasted its record compares quite favorably with the "reaction breakwater," increasing the depth during its construction period nearly 5 ft. When work was begun on the "reaction breakwater," the channel depth was 10 ft.; at the close of the construction period in 1896 there was 6 ft., a net loss of 4 ft.—a showing, thus far, quite in favor of the Mansfield Jetty. The principal mistake in the construction of the latter was not so much in design as in material.

The above loss of 4 ft., instead of the predicted gain of 15 ft., at the close of the work on the "first part" of the "reaction breakwater," indicated its failure as such, because its theory did not permit any such temporary obstruction.

According to this theory, since the work was begun on the outer slope of the bar, scour should have begun at that point where it would have "been assisted by gravity and the advance of the bar seaward prevented." Between August, 1895, when the breakwater

* Professor Haupt, in *Proceedings*, Amer. Phil. Soc., Vol. XL, 1901, p. 5.

† Professor Haupt, in his discussion, p. 343.

‡ Report Chief of Engineers, 1885, p. 1464

was begun, and August, 1896, when the "first part," which was to give 15 ft., was practically completed, the fill, at the exact point where the reaction should have begun and the bar "displaced laterally" by the helical action of the current, was just 7 ft. Maj. Gillette

As one of a pair of jetties, this result was temporary and to be expected. The scour beginning inside heaped the sand ahead of it on its way to the sea, and it formed a temporary obstruction. This obstruction happening to be nearly over the location of the end of the old jetty, then 7 ft. under the sand, the latter was blamed for the result.

"The channel depths increased rapidly until, in November and December, a channel of 13 ft. could be traced across the bar."

"Unexpected obstructions. It then transpired that the remains of the old Government jetty, which was reported to have 'disappeared,' were still in place covered with rock, crossing the bed of the channel and intersecting that portion of the curved reaction breakwater then in place about its middle point. * * * It thus acted as a submerged mat or retaining wall to prevent further scour, and as the breakwater subsequently rose, by the deposition of rock, to a plane three feet above the surface a perfect *cul de sac* was formed for the accumulation of sand. The harbor company was strenuously urged to remove the obstructing jetty, the existence of which was not suspected, as soon as discovered, but as it had made no provision for this unexpected work, either financially or in the contract, it was not removed. In consequence a shoal formed reaching to within 6-1/2 feet of the surface."

If there was a channel 13 ft. deep across this old jetty a few months before it shoaled at the same point to 6 ft., it is impossible to conceive how the old jetty should have formed a *cul de sac* to hold the sand 7 ft. over its crest, as stated, especially in view of the following statement of one of the company's constructing engineers, concerning some dynamite work that had been done there by Mr. C. P. Goodyear:

"For this reason the speaker does not think that the stone from the foundation of the old Government jetty which was scattered by the dynamite explosions prevents scour. On the contrary, if that stone is as well scattered as he believes it to be, he thinks that it is a help rather than a hindrance, on account of the local scour produced."*

A simple explanation of the phenomenon is shown on Fig. 51, which gives the profiles of each line of best water, at the beginning of work, at the close of the "first part," and a year later.

This shows that the sand scoured from the inner part of the channel was pushed ahead, forming a temporary obstruction. The location of the old Government Jetty, where the line of best water

* Thomas D. Pitts, Jun. Am. Soc. C. E., in *Transactions*, Am. Soc. C. E., Vol. XLII, p. [illegible].

Maj. Gillette.

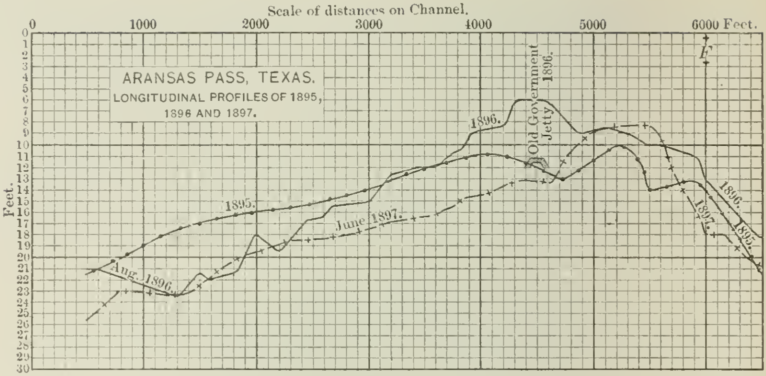

FIG. 51.

crossed it in 1897, is given in the figure and shows the absurdity of supposing that it held the sand bank in place.

Pushing the Bar Seaward.—Perhaps the most important claim made concerning the theory of the "reaction breakwater" is that it does not push the bar seaward. Aransas Pass is claimed as an example of the successful operation of this part of the theory, but

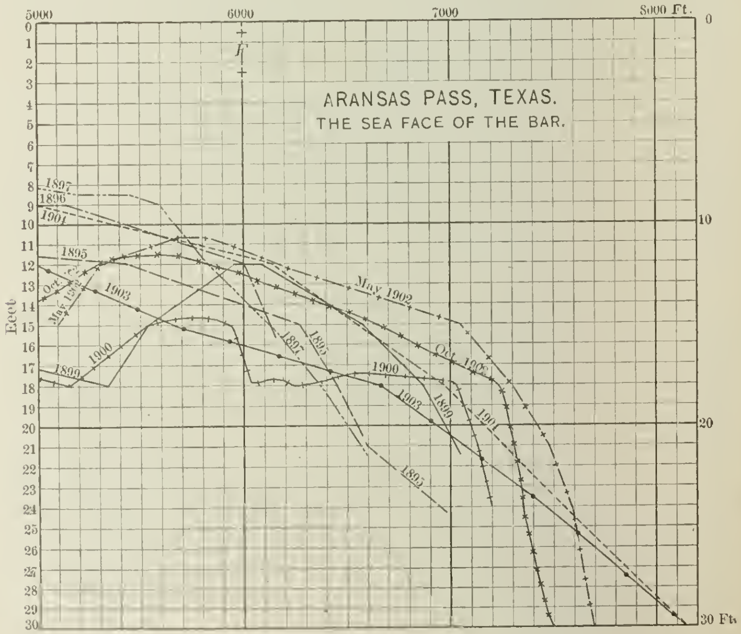

FIG. 52.

this claim will hardly stand examination. Presumably, if it has pushed the bar seaward, it would indicate the action of twin jetties and not of the "reaction breakwater." Mr. Gillette.

"The bar had not advanced seaward (1897) but has eroded on its inner scarp, which is a great desideratum in this class of work. Now (February, 1899), the 12-foot contours are cut through and but a few hundred feet separate the 15-foot contours," etc.*

This statement is correct, as far as it goes, but it fails to state that the erosion of the inner scarp had not at the time moved the bar seaward because it had heaped up the sand on top of it, and that

FIG. 53

when this was scoured out and better depths obtained the material was carried out to the outer scarp and deposited, thus advancing the bar decidedly seaward, as shown by Fig. 52. In any case, according to the theory, the outer scarp should have eroded, not the inner.

Fig. 52 shows the movement of the face of the bar. The sections are all taken on the same line, parallel to the outer part of the breakwater, *"A-F,"* and 250 ft. from it.

The movement of the contours seaward has not been as marked at this point as they have at other twin-jetty improvements—Cumberland Sound, for instance—simply because the results accom-

* Professor Haupt, in *Proceedings*, Amer. Phil. Soc., 1899, p. 140.

Maj. Gillette.

plished have been less. Aransas Pass is a small entrance, the width between the jetties is small, the outgoing flood volume is small, and the jetties are imperfect. The amount of sand removed to create the present useless trench along the "reaction breakwater" is very small and would probably not account for the movement shown in Fig. 52, were it not aided by the sand drift from the south. As this sand accumulates there will be plenty of "movement of the bar seaward."

To illuminate further the effects of the various works at this place, Table 23, showing the sand movement, has been prepared. The areas are indicated by Fig. 53.

TABLE 23.—Sand Movement, in Cubic Yards, at Aransas Pass, Texas.

Area.	1895–1896.	1896–1897.	1897–1900.	1900–1902.	1902–1904.	1895–1904.
A............	....	....	....	+33 000	+21 400	+101 400
B............	....	....	....	+130 000	+168 900	+312 900
C............	+100 000	—100 000	—47 000	+72 000	—45 600	—15 600
D............	+30 000	—70 000	000	+94 000	+40 900	+96 900
E............	+78 000	—130 000	—25 000	+25 000	+23 300	+3 300
F............	+190 000	—150 000	—180 000	+46 000	+60 000	+45 000
G............	+50 000	—42 000	—270 000	+200 000	+29 000	—13 000
H............	....	....	+52 000	+85 000	+1 300	+116 300
I............	....	....	....	+120 000	—38 900	+104 900
K............	....	....	....	+120 000	—43 300	+87 300
L............	....	....	....	+27 000	—39 200	+131 200
M............	....	....	....	....	+23 000	+145 000
S............	....	....	....	....	+98 300	+208 300
R............	....	....	....	—635 000	—51 500	—1 956 500
Total, south of breakwater, fill............						333 900
Total, north of breakwater, scour............						1 956 500

Area.	1881–1885.
S............	+55 000
T............	—166 000

Note.—Areas *E*, *F* and *G*, 1909-1902-1904, show fill in southern portion; there was scour in deep parts, next the breakwater.
+ Indicates fill, and
— Indicates scour.

Table 23 shows:

1.—That the area at the inner end of the jetty channel has shoaled considerably. This is one of the occasional defects of twin jetties. It has no place in the theory of the "reaction breakwater."

2.—That the area, "C," has scoured continuously for the last three years. There being no "reaction" along the convex curve here, this must have been due to twin-jetty construction. Maj. Gillette.

3.—That the area seaward of the channel has shoaled extensively. This must "push the bar seaward."

4.—That in the aggregate there has been an extensive fill of 1 333 900 cu. yd., south of the breakwater.

5.—That there has been a scour of nearly 2 000 000 cu. yd. north of the breakwater.

These last two facts are conclusive, both as to the direction of the sand drift and the true action of the breakwater.

If the sand drift were from the north, the 2 000 000 cu. yd. that have gone from the north of the structure would not have gone at all. Instead of a scour, there would have been a fill. However, assuming for the moment that it has gone south, owing to a drift from the north, there are only three possible routes for it, *viz.:*

First.—Around the end of the breakwater;

Second.—Through the gap between the breakwater and the shore;

Third.—Over the breakwater itself.

So large a quantity could not have gone around the end of the breakwater or through the gap without making radical variations in the depths and configuration on the bottom at these places, but no such variations appear on any of the maps. A small gradual fill is generally shown at those places with only slight changes in the contours.

It could not have gone over the breakwater without filling it on the north side to its crest or to the bottom of the short gaps in it, which it has not done at any time.

It should be noted that if the claim that the littoral drift were to the south were true, the absence of any stored sand north of the breakwater would condemn the structure as a failure, for it certainly has not "arrested the littoral drift."

The simple explanation of the matter is that the area of the bar to the north of the breakwater, as soon as protected from the ebb forces by the building of that structure, was promptly beaten shoreward and northward by the southeast waves, and the material is distributed along the beach of St. Joseph Island for some distance to the northward. The accumulation to the south is accounted for by the littoral drift from that direction being checked by the breakwater and the outflowing current. The only way for the sand, as it accumulates, to pass on north will be to do what is always done in such cases, *viz.,* to build a bar around the entrance and travel on it to the north. When this occurs the entrance will be in a condition very difficult to improve.

Maj. Gillette. Before the recent work of raising the outer end of the breakwater was begun, the work here was in a condition strikingly analogous to that of the Cumberland Sound Jetties, when the work described in the writer's paper was begun.

First.—In each case the outer portion of the windward jetty (the north jetty at Cumberland Sound; the south at Aransas Pass) was incomplete and inefficient to hold back the sand drift.

Second.—In each case the channel had been driven by the drift across the foundation extension of the lee jetty. At Cumberland Sound this foundation had been artificially breached to permit the use of this temporary channel. At Aransas Pass only occasional small boats came in, and they could easily cross the structure.

There the analogy ends. The first work done at Cumberland Sound was to stop the drift. This required both the outer and inner ends of the windward jetty to be raised, which was promptly done. This stopped the sand drift, and made a certain amount of concentration between the jetties. Much additional concentration was produced later, by closing the main ebb gap in the south jetty. At the time the above work on the north jetty was begun, the cyclical change of the channel due to the usual causes had started the new breakout about 1 000 ft. south of the line of the north jetty. This "psychological moment" was seized by doing the above work.

Professor Haupt refers to an article in the *Engineering Magazine,* of May, 1903. The writer of it apparently thought the success of this Cumberland Sound work was in some way due to an unrecognized adoption of Professor Haupt's theories. Nothing could be further from the truth. There is no "reaction" and no detached breakwater, while there is a substantial, necessary and effective second jetty.

True, the windward jetty was completed first, which is possibly somewhat analogous to the theory of the "reaction breakwater" if not to its unfortunately located "example" at Aransas Pass. The solid high-tide jetty at Cumberland Sound was planned to stop the littoral drift, and does it. The detached-breakwater idea plans to do the same thing, but it would fail. Had only the outer end of the north jetty been built, either straight or curved, the channel would now be only a fraction of its present size, and the great volume of sand drifting in through the gap would in a few years have driven the channel off to the south, to its old line across the south jetty.

The jetties are on the lines originally laid down by General Gillmore in 1879, eight years before Professor Haupt evolved the first of his remarkable theories. The work was done under the writer's plans, and strictly in accord with the principles stated in the paper under discussion. The success of the work does not ap-

pear to be due to anything but the application of those principles, and cannot, in any degree, be attributed to any extraneous or inadvertent causes. The plans were authorized only after thorough discussion and considerable opposition from others connected with the work. Maj Gillette.

To have applied to Aransas Pass, two years ago, the principles which have been found successful at Cumberland Sound, it would have been necessary, first, to build up and extend the old Government Jetty. A new line further south would now have to be adopted, since the recent work on the outer end of the breakwater, or "lee" jetty, prevents the proper line being occupied.

Unlike Cumberland Sound, the inner end of the windward jetty would not need building up, as it consists of the revetted head of Mustang Island.

The building of the outer end of the north jetty or "reaction breakwater" would probably have been unnecessary for many years, if the south jetty, instead, had been built up properly two years ago, before so much sand had drifted from the south into the region that must be included between the jetties, when the entrance is properly improved.

Professor Haupt says of Cumberland Sound:

"With the large bank lying between the jetties and the 'enormous sand bank which always moves very positively in one direction,' it would seem that the windward jetty has not yet completely controlled the littoral drift and must needs be extended."

The "enormous sand bank" expression, he quotes, is from the writer's report on Brunswick Bar, and refers to conditions at that harbor. The bank at Cumberland Sound between the jetties was there before the work was begun, and has nothing to do with the windward jetty controlling the littoral drift. This is clearly stated in the paper under discussion. No appreciable quantity of littoral drift has crossed the line of this jetty since it was completed.

It is to be noted, in this connection, that the sand drift from the south at Aransas Pass is obstructing the ebb so badly that a serious breach has occurred near the inner end of the breakwater (see Figs. 46 and 50). This, by the way, cannot be accounted for by "reaction."

Should this continue, which is not impossible, and the channel move into the area of erosion north of the breakwater, the condition of the port would be even worse than it is now.

Conclusions.

The foregoing discussion, as far as the writer knows, covers all theories of Professor Haupt that are available to the public.

Maj. Gillette. From it the following conclusions seem reasonable:

First.—That Professor Haupt's theories, promulgated in 1887-88, have little or nothing to do with his present "reaction breakwater" theory, and, in addition, are almost wholly erroneous.

Second.—The "reaction breakwater" theory has never been practically tested.

Third.—The theory is so palpably wrong as not to be worth such test.

Columbia River.—In closing, it may be well to devote a moment to the consideration of Professor Haupt's proposal for deepening the Columbia River Bar, shown on the very interesting photograph (Plate XXXII) which he submits. The breakwater this time is on the "windward" side, but the fatal gap shows on the photograph.

The structure was to be "so placed as to receive practically the whole of the effluent discharge." A glance at the picture will show that this cannot possibly be true.

Its curvature is determined by the curvature at Point Ellice. The analogy seems a trifle lame. The creating agency on the bar is to be shaped like the result produced by a rocky headland 10 miles inside. The other creating agency, Point Ellice, is not shaped a bit like the breakwater, and there is nothing to show that it "also operates to maintain" a channel with a certain curvature several miles below, or that it has "transported a portion of the eroded material to the counterscarp, some ½ mile away."

To the writer, it looks as though Point Ellice simply caused the current to dig a hole at its toe, as any bluff obstruction does; that the "counterscarp" was formed of detritus brought down the river by the current across the shoal, and that the curved channel below was the result of several forces. It does not seem logical to expect certain results on the bar from a "curvature" found inside, where the forces are wholly different. If it is, then why not take the deep channel shown at Astoria, where the curvature is much greater than at Point Ellice, or that near Point Adams, where the counterscarp is practically straight?

It should be noted that the gap shown in the photograph would not wholly exist in the actual construction. While the proposal indicates that the construction work should begin at the outer end of the breakwater, and work landward, a necessary preliminary, not shown in the picture, calls for the construction of a pile trestle 4¼ miles long built seaward from the outer end of the present jetty to the outer end of the proposed breakwater, the work to be done at $8 per running foot of trestle and $2 per ton for the rock necessary to hold the trestle in place.

As much of this trestle would have to be not less than 58 ft. above the bottom, the assistant engineer who has worked for years

Maj. Gillette.

on the bar estimates that an enrockment of not less than 16 ft. would be necessary to hold it in the face of the tremendous seas at this point; and that this would take about 700 000 tons of rock (about 15% more than it took to build the present jetty), making the cost of this preliminary work about $1 600 000, or two-thirds of what Professor Haupt estimated for the entire work, and that it would take at least three years to build it. If it took a longer time and more rock, the Reaction Jetty Company would have little to worry over, because it would have had a neat profit of more than 75 cents a ton on all rock put in, and in the meantime its prestige would not be much lessened, for it would be constructing, without risk, a very robust south jetty of a quite conventional type with a natural "twin" over at Cape Disappointment.

However, while this would doubtless have caused a temporary deepening, it would be very badly located, and would be an incubus on the entrance for many years after the "reaction breakwater" and its theory had become ancient history.

AMERICAN SOCIETY OF CIVIL ENGINEERS,

INSTITUTED 1852

TRANSACTIONS.

INTERNATIONAL ENGINEERING CONGRESS,

1904.

ENGINEERING EDUCATION.

Congress Paper No. 16.

By Robert Fletcher, Ph. D., Assoc. Am. Soc. C. E., Hanover, N. H., U. S. A.

Congress Paper No. 17.

By Calvin M. Woodward, A. B., Ph. D., St. Louis, Mo., U. S. A.

Discussion of the Subject by

J. L. Van Ornum, St. Louis, Mo., U. S. A.
D. H. Ray, New York City, U. S. A.
Frank O. Marvin, Lawrence, Kans., U. S. A.
H. N. Ogden, Ithaca, N. Y., U. S. A.
Alfred Chatterton, Madras, India.
Sir William H. White, London, England.
Robert Fletcher, Hanover, N. H., U. S. A.
Calvin M. Woodward, St. Louis, Mo., U. S. A.

Note.—Figures and Tables in the text are numbered consecutively through the papers and discussion on each subject.

TRANSACTIONS

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INTERNATIONAL ENGINEERING CONGRESS,

1904.

Paper No. 16.

ENGINEERING EDUCATION.

By Robert Fletcher,* Ph. D., Assoc. Am. Soc. C. E.

Engineering education during the past decade presents so many aspects and relations that any adequate treatment within the prescribed limits is impossible. In its broadest sense the term includes technical education of all grades. The facts and statistics relating to so wide a field are so varied and scattered, and often so far out of reach, that both public functionaries and duly appointed investigating committees obtain, by much labor, only incomplete returns. This paper will consider chiefly the status and work of the "engineering schools" of so-called professional grade, with some necessary allusions to others in some way related. It is presumed that other writers assigned to this subject will deal more particularly with technical education of secondary grade, in its relation to the wide range of technology in general, and that the general topic will be viewed from a European standpoint, thus enlarging our range of observation.

The present cannot be understood apart from the past. The progress of a decade is the outcome of preceding decades. The foreground of the picture covers the period since the Columbian Exposition of 1893. To give this proper perspective we need a background of about a century.

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* Director, Thayer School of Civil Engineering, Dartmouth College; Past President Society for the Promotion of Engineering Education

Briefly, such a retrospect may recognize three periods: First, the first third of the nineteenth century leading up to the railway era. During that time, with one exception, organized technical instruction was given only on the continent of Europe. Second, about thirty-five years, between 1835 and 1870, when government action and private munificence began to establish such education on many foundations. Third, about twenty-five years, to 1895, a period of phenomenal expansion, both as to the increase in number of technical institutes and colleges and the enlargement of older institutions. Other distinguishing features of these periods will appear as we proceed.

Viewing the first period when the dominating personalities of the engineering world, past and present, were Perronet, Brindley, Smeaton, Watt, Telford, Rennie and others of imperishable name, we see technical education obtained chiefly through the school of experience—the process of slow apprenticeship and cautious tentative methods. In 1804 systematic engineering education had but a small and scattered existence. In France the École des Ponts et Chaussées had been in effective operation less than half a century; the famous mining academy in Freiberg about forty years; the École Polytechnique, ten years; England had no technical schools; and in the United States the Military Academy at West Point had sent forth, just two years previously, its first graduate, destined to become a noted civil as well as military engineer.*

The great English engineers, led by Smeaton, had only recently compelled a recognition of civil engineering as a distinct profession differentiated from military engineering with which it had been commonly associated. In 1800, James McHenry, Secretary of War, writing to President Adams, thus defined the duties of the United States Engineer Corps: "We must not conclude that the service of the engineer is limited to constructing fortifications. This is but a single branch of the profession; their utility extends to almost every department of war; besides embracing whatever respects public buildings, roads, bridges, canals, and all such work of a civil nature."

Loammi Baldwin, a graduate of Harvard College in 1800, who has been styled "the father of civil engineering in America," was

* General J. G. Swift.

just beginning his career. He was identified with nearly all the greater works of internal improvement in the United States for thirty years. During this time, also, arose a line of celebrated inventors and self-taught constructors, some of whom had a fair general education and some very little. These gave to the world the first practicable steam navigation, the notable types of timber bridges—some of unprecedented spans (the trusses of Burr, Wernwag, Long, the Town lattice and Howe—wood, or wood and iron combined), and examples of canals, with "inclined planes" or "portages"—all achieved with singular economy out of inadequate resources. The lack of engineering education was imperfectly offset by information obtained in Europe, reports of scientific societies, etc.*

In those days auxiliary power on works was restricted to what could be realized from horses and gangs of men working in the treadmill, hauling on the lift ropes of pile-driving machines, etc. As late as 1830, in the construction of the Charlestown dry dock, "the pile-drivers were worked by a treadmill, although some objection was made to putting the free-born Americans into a machine which had so unsavory a reputation."†

The late J. E. Watkins,‡ Assoc. Am. Soc. C. E., mentions by name fifteen distinguished American engineers and twenty-six canals, railroads and other works on which they were engaged between 1812 and 1845. He also states that of 572 graduates of the United States Military Academy, from 1802 to 1829, 49 were chief or resident engineers on railways or canals previous to 1840. Of these 12 are named in connection with more than 20 engineering works and lines of professional duty. Also that from the report of the Chief of Topographical Engineers in 1835, it appears that 5 of the 10 officers were assigned to duty not connected with the army, and that, of 35 projects on which the Corps were then engaged, 15 were canal and railway surveys in the various States.

* As early as 1789, and later, in 1824, citizens of Pennsylvania formed a Society for the Promotion of Internal Improvements. In 1825 William Strickland, Civil Engineer, was sent to England in their interest. His reports, published in folio in 1826, with plates, describe the typical canals, locks, bridges, roads, tunnels, tramways with cast-iron rails, iron furnaces, coke ovens, breakwaters and other harbor works; also one of Stephenson's patent locomotives, for that was the period of experimentation before the great first triumph of the new motor.

† Geo. L. Vose, C. E., "Sketch of the Life and Works of Loammi Baldwin," p. 17.

‡ "The Beginnings of Engineering," *Transactions*, Am. Soc. C. E., Vol. XXIV, p. 309.

In 1825 more than 1 200 miles of canals had been built, the Erie canal having been begun in 1817.

Among other unrelated beginnings this period includes the establishment of the Polytechnic of Vienna in 1815; the Royal Polytechnic of Berlin in 1821; and the Rensselaer Polytechnic Institute of Troy, N. Y., between 1825 and 1835.

We must notice the influence of the European schools on two continents. From early in the century they were the source of the principal engineering literature, and were resorted to by American students. It must not be forgotten that the progress of engineering science in America from 1775 to 1825 was to a considerable extent directed by Frenchmen who came to this country by the request of the Government. Five were appointed by President Washington in 1794. The United States Government sent Colonel (then Lieutenant) Sylvanus Thayer on a tour of inspection, to examine, among other matters, the military and other technical schools of Europe. The scheme of study, discipline and administration which he developed as reorganizer and Superintendent of the United States Military Academy, from 1817 to 1833, showed the influence of the French ideas and scientific methods of that day. As to textbooks, the French treatises on mathematics, physical science and the art of construction were generally preferred because of their singular excellence in clear statements of definitions and principles, adequate illustrations (expensive in those days) and terse but comprehensive presentation of the subjects.*

An incident in this connection is related of the late Charles S. Storrow, Hon. M. Am. Soc. C. E., who studied in the two leading schools of Paris. Just after graduation from Harvard College in 1829 he was asked by Dr. Channing the theologian, "What is civil engineering?" He replied with enthusiasm, describing the studies and qualifications needed to make a civil engineer, when he was interrupted thus: "Hold on, Charles, you have told me enough to convince me that no one man can ever become a civil engineer." However, the young man with the clear vision disproved this hasty judgment, and, equipped with his thorough education, became the

*Sganzin's "Programme of Civil Constructions" at the Polytechnic, Paris, was translated (inadequately) into English and thus published in Boston in 1827. We may mention here also Girard's "Grands Routes," Treussart on Mortars, Vicat on Mortars (1818), Tredgold's Carpentry, Barlow on Strength of Timber (1824); De Pambour on the Locomotive, Wood on Railroads, and various French works on mathematics, physics and hydraulics.

successful early railroad builder, the creator and manager of the great hydraulic works of the Essex Company at Lawrence, Mass., Consulting Engineer for the State on the Hoosac Tunnel, etc.

But a little later, Europe was indebted to America, and reaped the fruits of sound technical training. A Commission from the Russian Government, after investigation on both sides of the Atlantic, selected Major G. W. Whistler, a graduate of the United State Military Academy, as best fitted, by his experience, knowledge and ability,* to design and build the railway from St. Petersburg to Moscow (1842-49). His report on the long debated question of gauge and other minor details of this great national work is said to be "one of the finest models of engineering argument ever written."

Coming to our second arbitrary period we note a few landmarks. The first graduates in civil engineering in the English-speaking world came from the Rensselaer Polytechnic Institute in 1835; in England the first formal teaching in civil engineering began in University College, London, in 1840; the same year Queen Victoria founded the chair of civil engineering in Glasgow University; here Rankine, eminent both as practician and teacher, developed the strongest type of engineering education based upon sound mathematical theory, and gave to the world those profound and masterly textbooks which had a world-wide influence in the schools for two generations. In 1845 the School of Engineering in Union College, N. Y., was started; in 1846 the Lawrence Scientific School of Harvard University; in 1847 the first engineering college in India; in 1851 the Chandler Scientific Department of Dartmouth College; and in 1852, the Engineering Department of the University of Michigan. Between 1860 and 1875 the following, among others, were started: Sheffield Scientific School of Yale (as to engineering courses); Columbia College School of Mines; Massachusetts Institute of Technology; Engineering Department of the University of Illinois; College of Civil Engineering in Cornell University; Worcester Polytechnic Institute; Stevens Institute of Technology, a school of mechanical engineering solely; Washington University,

* Whistler had been Chief or Consulting Engineer on the pioneer railroads in Massachusetts, of the Baltimore and Ohio Providence and Stonington, etc., Director of the construction of the first locomotives for these roads, and was considered the leading railway expert of his time: trusted for his keen insight, sound judgment and high integrity, fortified by great ingenuity and skill.

St. Louis, Polytechnic School; Thayer School of Civil Engineering at Dartmouth College; Lehigh University, Department of Engineering; Engineering Department of the University of California; Washington and Lee University, Engineering Department; Sibley College of Engineering, Cornell University; Schools of Engineering, Purdue University; and Rose Polytechnic Institute.

Some of these institutions are memorials of the generosity of public-spirited individuals, with resources gradually accumulated; others result from the wise foresight and truly conservative policy of State legislatures; and those commonwealths have reaped a rich return of power and wealth consequent upon the diffusion of sound scientific knowledge and technical skill directed to useful ends. The general purpose of such is well expressed by the terms of one donor in establishing a school of science, more than fifty years ago, as follows:

"For the establishment and support of a permanent department or school of instruction, * * * * in the practical and useful arts of life, comprised chiefly in the branches of Mechanics and Civil Engineering, the Invention and Manufacture of Machinery, Carpentry, Masonry, Architecture, and Drawing, the Investigation of the properties and uses of the Materials employed in the Arts, the Modern Languages and English Literature, together with Bookkeeping and such other branches of knowledge as may best qualify young persons for the duties and employments of active life."

Some of the above institutions, and others not named, are the so-called "land-grant" colleges made possible by act of Congress in 1862; their organization proceeded slowly in some States, and is one of the features of the expansion in the third period.

A note should be made of the early reciprocal educational influence which proceeded from America through native textbooks. The first distinctive treatise on "Civil Engineering" in English was written and published in 1837 by Professor Mahan, the distinguished Professor of Military and Civil Engineering at the United States Military Academy during thirty-seven years. This was republished in England, was used as a textbook in India, and was translated into other languages.* In 1835 Mr. Storrow published

*In 1872 fifteen thousand copies had been sold by the American publishers. Mahan's "Stereotomy and Stone Cutting," "Industrial Drawing" and three works on military engineering were widely used as standard for half a century. Other noted textbooks from the Military Academy were: Davies' series of mathematics, Church's works on higher mathematics and descriptive geometry, etc.

his "Treatise on Water-Works for the Conveying and Distributing Supplies of Water." This was probably the first systematic treatise on this subject in the English language. It is a small octavo of 212 pages, with four excellent plates, based upon the writings and experiments of Prony, Belanger and Genieys, and some British and American scientific memoirs, and was a work of great merit.

During the first part of our middle period there was no marked increase in the number of schools. In 1866, in the United States there were 6 of recognized standing; but during the next five years the number increased to about 21; meanwhile the aggregate of graduates increased from about 300 to probably less than 900. There was slow growth at best, in some cases almost none; and this was so far separate from the profession which they aimed to serve that the methods and subjects of instruction were not properly adjusted to the demands and opportunities of practice. The great masters of mathematical exposition, like Navier, d'Aubisson, Bresse, Rankine, Mosely, Bartlett at the United States Military Academy, etc., set a fashion of teaching which was criticised as too severe and impractical. The methods which wrought such marvellous results in celestial mechanics were applied to the entire range of "natural philosophy," so-called. School facilities being inadequate, and class laboratories almost unknown, the results of experimental research were dressed in mathematics and taught largely through great "weight of authority" from the book; and, while the instructor performed elementary experiments on the other side of the table, the class simply "observed" and perhaps tried to "take notes." Adverse critics thought that applied mathematics had become to a great extent mathematics misapplied. The extreme statement of one honored practician was that some master-minds "exhibiting a profundity of knowledge beyond the reach of ordinary men" had contrived to "bury the most simple facts out of sight under heaps of mathematical rubbish." Such criticism is worthy of notice only as showing the mistaken attitude of a certain class of "practical" men toward the investigators; it betrays ignorance of the triumphs of research which brought forth the science of thermodynamics, established the mathematical theory of magnetism and electricity, etc. However, a happy mean between these extremes was gradually realized; the value and necessity of the mathematical

training was too evident for successful dispute; the engineer's demand for utility has made mathematics properly subservient as a tool—a means to an end; demonstrations have been simplified, non-essential discussions excluded; proper co-ordination made with the data of experiments and all the material of instruction; so that, in substance and form, the better recent textbooks and manuals for American students are well adapted to the practical ends in view, and accepted as standard as well without as within the limits of the schools.

A loud criticism of the schools, which was in vogue about thirty years ago, has thus been stated elsewhere.*

"It was claimed that engineering instruction was almost solely devoted to abstract principles; that it was largely misdirected because separated from the objects, phenomena and conditions of practise; it was, therefore, ineffective and usually contemned by the practicians; the spheres of the investigators, schoolmen, and men of engineering affairs were too wide apart and their labors not correlated; the young graduate was said to be nearly useless even as a conservator; a change was demanded which would result in a better adaptation of means to ends and make the graduate more immediately available."

We have characterized the third period as one of great expansion, both as to number and variety of schools. In 1892 a careful investigator gave the following statistics of 52 engineering schools of all grades.† Aggregate of graduates above 9 000; from 45 schools or colleges 5 400 had graduated as civil engineers; from 15 schools, 870 as mining engineers; from 20 schools, within three years, about 200 as electrical engineers; from 33 schools, 2 800 as mechanical engineers; this differentiation had been in progress about 20 years, dating back to the opening of the Stevens Institute, for mechanical engineers, to the Columbia College School of Mines, and to the Massachusetts Institute of Technology, for both courses.

Only three years later another investigation‡ put in evidence 109 institutions presenting courses of study in various lines of engineering.

The great demand which provoked and maintained this expan-

* "A Quarter Century of Progress in Engineering Education," by the writer, *Proceedings* of the Society for the Promotion of Engineering Education, Vol. IV.

† Editor of *Engineering News*. Series of articles on the "Engineering Schools of the United States." He named 94, but 42 were quite recent or sent no sufficient returns.

‡ Committee of the Society for the Promotion of Engineering Education.

sion was world-wide; increase of colonization and wide extension of commercial interests called for the exercise of all the knowledge and skill of the engineering profession; as to America it was said*:

"Railroads, bridges, water-works, sewerage-works, and mining and metallurgical plants were demanded over the face of the continent. Hundreds were crowding or were being pressed into service with little or no proper education. Chainmen and axmen speedily became transitmen and "engineers." To such practicians the defects of the schools were more obvious than their own deficiencies. Yet the value of technical training began to be realized by some who had only crude ideas as to what it should be."

Another phase of expansion which was also a great movement tending toward the required readjustment was the inauguration of shop instruction or "Manual training." In 1868 Della Vos began at the Imperial Technical School of Moscow class instruction in the use of tools—the aim being for instruction only, not finished products. Entirely independent pioneers in America began with a little different idea: C. O. Thompson started the shops at the Worcester Institute in 1868, with a commercial or producing end in view; Robinson started the shops of the University of Illinois about 1870; Thurston the laboratory and shops of the Stevens Institute about the same time, laying special emphasis on testing, and directing the shop work to the production of machines. The exhibit of the "Russian system" at the Centennial Exposition in 1876 aroused wide interest in America. President Runkle of the Massachusetts Institute of Technology became a strong expounder of the system, and started shops in 1877; C. M. Woodward started the "Manual Training School" in connection with Washington University, St. Louis, in 1879, and has constantly and consistently promoted this form of teaching ever since. It must not be forgotten that Girard College, Philadelphia, adopted the principle of manual training for artisans in its early history; nor that the most successful movement for the education of the colored race in America was based strictly on this idea, inaugurated at Hampton Institute, Virginia, and developed by Booker T. Washington at Tuskegee, Alabama. Of this system it has been well said:

* The writer's paper, before referred to.

"There has probably never been a movement in American educational methods which has effected such great changes in so short a time, as this manual training school movement."

Another great movement nearly simultaneous with that just noticed, is the rise and development of the German monotechnic schools. Realizing that even the most industrious people must have competent expert direction and that "efficient direction of any industry to-day demands a large amount of technical knowledge which cannot be learned at the bench or in the shop," the Government, and the people through trade associations, have established hundreds of schools of applied science for instruction in all the leading industries of the empire, and often many schools for the same industry. Six years ago the late lamented Professor J. B. Johnson, M. Am. Soc. C. E., reported that, of 248 monotechnic schools in Prussia alone, more than half were voluntarily supported by various trades, as schools for apprentices; in Saxony, with 1 000 000 inhabitants, there were 3 monotechnic schools, besides 10 schools of agriculture and 40 of commerce; in Hesse, schools of agriculture and sculpture, 9 for artisans, 43 for industries and 82 for design; in Baden, schools of architecture, industry, commerce, etc., and so on. And these are supplemented in Germany, Belgium, and France by colleges of commerce to prepare men for all forms of commercial activity and aggression. Whether or not this is overdone time will show, but the writer's comment then was:

"Germany has seen this situation most clearly, and her clear conception of this problem, and her rational and thorough solution of it, has raised her industrially from poverty and obscurity to wealth and fame in the short space of a quarter of a century. * * * It is not her army of soldiers which other nations need to fear, but her army of scientifically trained directors of industrial enterprises and of highly educated commercial agents."

At the close of this period, about 1890 and later, we may note some well marked and settled features of the educational situation. American students do not need to resort to European schools. At home the colleges and institutes for higher engineering education are entirely adequate and better adapted to meet all demands of the country; they partake of the spirit of free institutions; as has been well said, they work on the principle of the vertical uplift,

giving every worthy man free opening at the top; while in less favored lands society is stratified horizontally and few can rise above their class. However, each has some lesson to give to another. German and British engineers have conceded the greater effectiveness of American methods in the use of fully equipped laboratories and shops, and the value of the various forms of practical instruction given. Englishmen have deplored the insufficiency of the higher technical education in the very home of the great succession of British engineers, and have remonstrated at the large waste of funds in misdirected secondary instruction to artisans. American observers in Germany, while disapproving of the continued subserviency to the lecture system, to the exclusion of what they deem needful class work and laboratory instruction, find that America has need to learn the lesson of enforcing higher standards of fitness for "entrance," and a greater degree of thoroughness throughout the entire system. Again, the old distrust between "schoolmen" and practicians has been largely removed; in the ranks of the profession the "graduates" have been growing to a majority; practicians have been called to be lecturers and professors in the schools; and the schools in turn have given to the profession most important results of experimental research, and treatises and manuals everywhere received as authoritative; contractors and manufacturers extend all aid and courtesy to classes of students visiting their works for inspection and instruction; and in other ways a true community of interest is recognized.

Some of the foregoing considerations are obvious, familiar and elementary, but are pertinent and necessary to give the picture any degree of completeness.

The Past Decade.

In nearly all the aspects of technical education, continued activity and expansion characterizes both the recent past and the present. The number of institutions is still on the increase; older ones are strengthened and extended by the aid of princely donations; some are newly born to a struggling existence; some, established by private munificence, have had at the start very complete equipment of boards of instructors, shops and all needed machinery; educational experiments have been on trial; commercialism has

invaded the field; departments of research for colleges and institutes have done important work; methods, programmes of study, and older policies have been modified and are under discussion; and the end is not yet.

Not the least among the influences for the betterment of these interests in the United States, is the Society for the Promotion of Engineering Education.* Its membership has included leading educators and engineers of two continents. Two years ago its committee on statistics reported a list of 118 colleges and institutes of technology, two being in Canada, one in England, and one in Australia. Of the entire number, 35 are designated as State Colleges or Universities; 24 as State Colleges of Agriculture and Mechanic Arts; some under both names are supported, at least in part, from proceeds of the Congressional land grants, or annual allowances for agricultural experiment stations; the other half are incorporated colleges or technical institutes of all grades. During the year 1901-02 60% of the entire number (the others not sending adequate returns) enrolled about 17 200 in engineering courses, and about 13 500 males and 7 000 females in "academic" courses, one-tenth of the latter having courses in "domestic science and art." The extent of differentiation is seen in the classification by courses, with their respective enrollment, *viz.*, chemical (247), civil (2 886), mechanical (4 854), mining (1 740), electrical (2 867), metallurgical (56), sanitary (14), municipal (4), industrial and mechanic arts (378), architecture (389), naval architecture (77) and "special" (2 159). These figures are not sufficient for very definite conclusions, and may easily be misinterpreted, but they indicate the continual tendency toward diversity and possibly premature expansion in certain directions. About the same time the United States Commissioner of Education gave a statement of courses of study of 134 "universities, colleges and schools of technology."

Undoubtedly some of the privately endowed institutions are examples of unwisdom—especially when the funds are limited and other schools are available. Too many are unable to realize their high ideals. There is need of concentration all along the line.

* Organized at the Columbian Exposition largely through the zeal of the late Professor J. B. Johnson, its life is co-incident with the decade. At its annual conventions both the larger and lesser interests of all technical education have been fully discussed. Its committees have made investigations and reports of the highest value. Its volumes of *Proceedings* embody the matured opinions of influential and representative teachers and practicians.

Future generous donors should not be ambitious to build the new but seek out and strengthen the old existing colleges or departments. However, in new and growing States or colonial settlements, the best colleges are usually out of reach; needs are urgent; resources are limited; hence a beginning is made and heroic work is done; but such cramped and adverse conditions result in admitted shortcomings.

The higher education does not stand alone; amidst some complexity and apparent confusion in the situation, there is such necessary relation to other grades that the present aspect of the wider field must be considered.

Colleges of Agriculture and the Mechanic Arts.—A College of Agriculture and the Mechanic Arts is found in nearly every State of the Union, under some form or name. The report of the United States Commissioner of Education for 1902 names 50 such colleges for white students and 16 for colored students. In common they usually give a four years' course, approximating more or less to college grade, including the subjects implied by the above general title. In some States a short agricultural course is added.

"In Minnesota and in some other States * * * an 'agricultural school' has been correlated. This is a true secondary industrial school in which the art and science of farming are taught both boys and girls. It teaches suitable English studies, physics, chemistry, the raising of crops, the use of fertilizers, animal husbandry, dairying; cooking, sewing and household economy to the girls, etc. This course is wonderfully successful."*

But by far the larger part of the graduates turn away from the farms; they prefer some line of engineering practice, or become identified with manufacturing industries as managers, etc. Hence these are in effect an important class of engineering schools, giving a sufficient scientific and practical training for those grades of engineering which constitute a great part of the practice of to-day.

"They are near to the people, are comparatively inexpensive and are very largely attended. They have more than justified their cost, though most of them are not altogether fulfilling the purposes contemplated in the original enactments."

* Report of Committee on American Industrial Education to the Society for the Promotion of Engineering Education. New York meeting, June, 1900.

One State university has set an example of a "summer school" policy—a phase of university extension work—utilizing a part of the long vacation, to give short courses of instruction to apprentices, artisans, central-station men, etc. Courses are given in steam engines, and other heat engines, applied electricity, machine design, materials of construction, shop work and surveying. As the price of tuition has to be rather low on account of the limited means of those who are expected to seek such opportunities, the University has to make a considerable appropriation to cover deficiencies. The differences in age, capacity, previous preparation and ultimate purpose of the beneficiaries, introduce troublesome conditions. The motive may be said to be a "sense of duty" to a large public constituency, whose interest in the State institution is recognized in their providing for the needs of the wage earners on the side of technology. Whether sufficient numbers will take the opportunities offered, or can spare the six weeks of time required, and also meet the cost, is yet a question. While results are fairly satisfactory for a beginning, the plan may perhaps be said to be at the stage of a hopeful experiment; it serves but imperfectly as a monotechnic school of the German ideal, because of the shortness of time and the mixed clientele.

The State university has been urged to assume also the duty of educating teachers of sanitary science who shall be able to enlighten the people on questions vitally affecting the public health. Also to develop courses in "chemical engineering," to prepare managers and superintendents of large chemical industries—and this is being done by certain institutions; also to develop the higher commercial colleges. But the latter need is already otherwise provided for: the Wharton School of Finance and Economy at the University of Pennsylvania, founded in 1881, has been followed in sixteen colleges and universities by departments of "economics, commerce and finance" or "economics, commerce and industry," etc., including the recent establishment of a post-graduate "School of Administration and Finance," at Dartmouth College, on a $300 000 foundation.

Correspondence Schools.—These schools may be said to have passed beyond the period of novelty. Although in evidence ten-

years ago,* their great expansion has been within the past eight years. To give a clear view of the facts necessary for a just estimate, within limited time and space, is difficult. These schools seem to offer means of instruction to workers of all ages and pursuits, whose conditions of life bar them from the opportunities offered by the higher and secondary institutions. They claim to supply the needs of the people; to teach a man more about the work which is his own special concern; to increase his efficiency therein; to give him that knowledge which is power, and make him worth more wages. The leading facts as to development are: One, devoted chiefly to engineering and mechanics, had 80 000 enrolled students five years ago, this being a four-fold increase within two years; four years ago the oldest one had more than 200 000 names on its rolls; at the present time one is said to use 100 000 printed "form-letters" per month; in 1902 the United States Commissioner of Education reported in regard to one of the largest and apparently the most flourishing:

> "The advertising matter of this institution appears extensively in all the principal magazines and periodicals, and it has over twenty branch offices, and employs regularly a force of 2 500 persons in all its departments. Over 350 000 students were enrolled in the first ten years of its existence, but for actual instruction only 26 'principals,' with a total of 358 'experts, instructors and assistants' were required in 1902. Thus it will be seen that the greater part of the receipts from tuition go for securing and retaining students, rather than for instructing them."

Within six years, six leading schools in as many different cities have advertised in the engineering journals, besides two others of lesser note and more restricted scope. One of these had for its president an engineer educated in the best institutions of two continents, who held foremost rank in his specialty. Six years ago one had erected a home building at a cost of $225 000 and offered nine separate engineering courses (civil, mechanical, mining, municipal, bridges, hydraulics, railway, electrical, surveying and mapping). Engineers of high standing have been enlisted to prepare series of textbooks of great merit in the points of clear but abbreviated treat-

* The plan was first an outgrowth of the Chautauqua School in 1880, and was for a time in extensive operation for academic work, but the "Chautauqua University," incorporated, was not financially successful, and was discontinued after ten years of trial.

ment of the subjects, good typography, the best of illustrations and timeliness in being fully up to date. Lower-paid men, sometimes of questionable ability, give more or less adequate scrutiny and corrections to the thousands of instruction papers sent in by the learners. One great school, conducted in connection with one of the largest and best technical institutes advertises to give technical instruction by mail in about forty of the leading trades, factory employments, railway positions, etc., up to civil and mechanical engineering; it announces by name nine leading professors and practicians who prepare the instruction papers for home study; and it conducts a monthly periodical in its interest. At least two American universities of the first rank have adopted this mode of instruction, but not for engineering subjects. President Harper testifies that the best students do better work in some subjects by correspondence than in the classroom; but no degree is given unless a year is spent as a resident student. Twenty-six subjects were thus taught in the year 1901-02.* As to methods and operation—thousands are induced to enroll though specious and often misleading advertisements or the interested agency of soliciting agents; a large part of the students lack in aptitude or definite aims, ability or patience to persevere, and, after paying for a course, which usually involves the purchase of a set of books, become discouraged and drop out; statistics of one school showed that in one group a total of 70% had completed no subject beyond arithmetic, and that only five out of a thousand had "graduated;" in the nature of the case, the system is defective in that the courses lack breadth and depth, are without the stimulus of class-association and personal contact with instructors, and are without systematic instruction in shop or laboratory; the demands for the higher courses grow relatively smaller, and those who attempt them more generally quit, while the numbers taking the lower or trade courses grow relatively larger. As a business—the means of promotion are purely commercial; money-making is the chief end, and all the tricks of advertising and campaigning through paid "drummers" are resorted to; even instructors in regular colleges are urged to take courses for "special improvement," with offers of sets of textbooks in various lines to be had for the price of a course; the administrative methods in treatment of scholars are generally

* Report of the Commissioner of Education, 1902, Vol. 1.

good, but the sensational advertising and jealousy of other schools are detrimental; the best have adequate resources, are conservative and reputable; others give evidence of charlatanism. One competent critic thinks that, while they now do but imperfectly the work of trade schools, they will continue to retrograde and ultimately do that kind of work. Evidently only the most heroic and capable toiler or artisan can do effective studying after a hard day's work; hence there is a vast aggregate of mistaken effort and unfulfilled expectation. On the other hand, many, even of the majority who do not persist, get some mental uplift; others go farther, and gain knowledge and promotion; and a few, persistent and unincumbered, may contrive to take some "summer course" in residence, while less often the lesser aim is abandoned and the higher institute resorted to. One well-informed observer thinks that the correspondence school is a most important development of modern educational methods; another gives judgment that their graduates will be skilled workmen, not engineers, and that even thus these schools will do much good and are worthy of encouragement.

The great success achieved by the manual training system is already noticed, and the effective work done by other great schools, of which the Pratt and Armour Institutes are examples—richly endowed and equipped with laboratories, workshops and competent directors and teachers—has led to a further extension in the form of the manual training high school as part of the public school system of many large cities. Statistics are given for 124 "manual and industrial training schools" in the United States in the report of the Commissioner of Education for 1902, including all grades, whether sustained by public funds, private enterprise or philanthropy. We cannot here discuss the distinctive functions of manual training schools, manual training high schools, day trade schools, "evening technical and evening trade schools," corporate or "proprietary" trade schools, "city and endowed evening schools," half-time self-supporting trade schools, etc.; the titles alone speak of the diverse endeavors made by philanthropists and boards of education to devise opportunities for the rising generation and prepare for the demands which the rapidly increasing population and enormous expansion of industrial activities will continue to make upon all educational facilities.

This general view of the more subordinate interests, while showing great diversity of purpose and action, reveals an effectual striving toward a solution of the old problem, how to bring about an adequate "union between engineering science and art."

A general verdict on the work of the higher engineering colleges may be stated in borrowed language:*

"These are strictly professional schools of a high grade which teach both pure and applied science, and which rank with the best of their class in any country. In the training of professional engineers they are about all that could be desired; and they are now, and have been for many years graduating young men as well equipped in both theory and practise as can be found perhaps anywhere in the world."

This is a *tout ensemble* and has all the indefiniteness of such a view; we have noticed hindrances and deficiencies, and must not ignore other drawbacks and some debated questions of methods and aim.

In regions of the United States just awakening to an appreciation of their material resources, where general education has been slighted and manufactures neglected, where the industrial worker was formerly in disrepute and engineering education could get little public support, engineering colleges are at the rear of the procession and have much up-hill work to get into line abreast. However, in these, as in all other cases of scant resources, the really capable men under competent direction may and do grasp the fundamental principles and correct theory to such extent that the deficiencies in practical instruction are eventually made up, but with loss of time and effectiveness.

A near view of the present status and trend of engineering education may be had through a brief statement of some of the topics under lively discussion during the past five years or more.†

Entrance Requirements.—According to a report of a standing committee, after some years of investigation, the standard has been made very high, including a rather advanced stage of mathematics, a wide range of physics and chemistry and a considerable course in modern languages. There was some objection that the majority of

* Report of a "Committee on Industrial Education" before referred to.
† *Proceedings* of the Society for the Promotion of Engineering Education, Vols. V to XI inclusive.

preparatory schools could not thoroughly fulfil the conditions; emphasis was laid on the importance of proved aptitude and careful preparation of applicants; some thought it of less consequence to have uniformity on these, but would have more on the graduation requirements.

Minimum Graduation Requirements Proposed.—There were diverse opinions, because of variety of courses and time available; in some cases there were only four years between the secondary school and the degree, in other cases five. This involved the much mooted question of various and numerous degrees as against three or four significant titles. Inquiry by a committee revealed the fact that more than sixty different degrees have been conferred or offered by upwards of a hundred engineering colleges. But the tendency is toward avoiding the uncertainty and confusion due to so many titles, and greatly restricting the number, preferably to four or six college degrees and five or ten professional degrees (five at most, if the superfluous higher (?) title of Master or Doctor, proposed in some quarters, be not foisted upon the profession). There will probably be general agreement that it would be a mistake to depart from the simplicity and dignity of Civil Engineer, Mechanical Engineer, Mining Engineer, Electrical Engineer, etc., which terms should be broad and inclusive in their significance. It is urged that a uniform standard of graduation is most desirable, but, under the diverse conditions prevailing, it does not yet seem to be practicable, although an approach to it is well begun.

Value of Instruction by Non-Resident Lecturers, and Abuse of the Lecture System.—The consensus of opinion is in favor of the judicious use of lectures. The value lies in the personal inspiration and authority of the practician; the necessity, in bringing subjects up to date; the defect is in the violation of the principle that the student masters most thoroughly that which costs him strenuous effort.

Over Development in Engineering Laboratory Courses.—It is claimed that there is a tendency toward too much work in the laboratory, and especially lack of co-ordination between class-room and laboratory work. Each should be carefully proportioned and adjusted to its place.

Alleged Fundamental Mistakes of Engineering Schools.—(a)

waste of time by too much vacation; (*b*) too much classwork; more work should be done by and with the individual; (*c*) mixing of preparatory with engineering subjects. As to (*a*), it is judged that the waste is more apparent than real; many students get instructive professional or related helpful employment during vacation; instructors must have time for professional advancement, and their work is not all confined to term time. It is admitted that some vacations are too long and holidays too frequent. As to (*b*), large classes with few instructors are to be deplored. Students should be taught in small sections; more frequent and more constant contact with the instructor is needful, in the field, laboratory and drafting room. This is an important condition of efficiency. As to (*c*), the opportunity for improvement is admitted. It is unavoidable in some four-year courses with inadequate teaching force. If possible, strictly engineering studies should be massed in the last two years; in an exceptional case the preparatory studies are confined to three years, a fourth year with chiefly engineering studies leading to the B. S. degree, and a fifth year with exclusively engineering studies to the C. E. (or other) degree. With small classes and students working under close supervision, this is the ideal arrangement. Closely related to the above is the much discussed question following.

Excessive Differentiation, or Danger in Too Much Specialization.—Extreme views are held. Some would have the college programmes cover an all-around training in fundamental subjects only, grounding the student in the basal principles of mathematics, mechanics and physical science, and those which determine engineering practice in general, giving due time also to broader culture but giving only unavoidable attention to details. Others, arguing that this is above all things the age of specialists, would have the student begin to differentiate his studies with reference to some specialty, in view, even in the secondary school. Others claim that the more common arrangement is necessary and sufficient, that which devotes two or three years of a four-years' course to the more general underlying subjects, and the final two years, or one, to study and practice in the special direction. Others argue that the specialty is the lifework and should be begun last, and that the schoolwork therein is better done in a fifth or post-graduate year.

It is certain that many, heretofore, have turned to a specialty with undue haste, and have found later that it was a mistaken choice, and that they have narrowed their future possibilities; it is also certain that men with the large and more thorough all-around training have been able to adjust themselves to a wider range of emergency and opportunity; while there is force in the argument that in the broader curriculum it is difficult to treat the separate subjects with the necessary thoroughness, it is found in practice that one well trained in the fundamentals seldom fails to fit himself (given a little time) to a special responsibility. It has occurred in certain instances that the engineer as a practician and the same man afterward as a teacher has held quite different views as to what the school should undertake to do. The ideal engineering education has been often described in interesting papers and discussions; but in this matter-of-fact world the ideals are rarely verified —certainly not in the working conditions and environment of most of the colleges; not in the entire make-up of boards of instruction; seldom in the qualifications of the students, and, even if an occasional ideal graduate is produced, he does not often find at first-hand his ideal of professional opportunity.

A judgment of the engineering schools was sought, about four years ago, at the court of last resort; the following questions were proposed to some of the prominent engineers:*

1.—Does the number of graduates of engineering schools each year exceed the demand, in prosperous times?

2.—Are there superfluous and incompetent graduates? Do the older practicians regard them as unwelcome intruders who tend to lower the rates of compensation?

3.—What is your opinion of the average gradute, from the standpoint of the employer?

4.—Have you ever known graduates so well trained as to be trusted at the start in places of considerable responsibility?

5.—Do you think it expedient for the schools to go far into special departments of engineering; that is, do you think that they are anywhere undertaking too much?

* These were sent to 63 persons, of whom 47 replied, some of the latter being leaders in the profession. Some of the replies were at considerable length and indicated much interest in the subject.

6.—Will you frankly state what in your opinion are some shortcomings of the schools as to aim or method?

The briefest possible summary of the replies follows: Only about 20% conceded that there were many superfluous or unwelcome graduates; the others believed that the demand for capable men did not exceed the supply. About 75% held that the young graduates are not unwelcome, that they do not injure the business of older men—who in fact prefer to employ them—and that incompetents are no more in evidence than in other professions. Only 4% admitted that the business of the older practicians is interfered with by the new comers. Some added significantly: "The schools have made it impossible for untrained men to enter the profession." About 18% of the respondents had a high opinion of the young graduate, finding him generally well adapted to his work; about 65% a low opinion, with modifications to the effect of his having much to learn, but that he improves rapidly, knows too little and that not well, etc.; the others were non-committal. As to specializing, about 22% thought the policy radically wrong, as tending to a trade rather than a profession; about 33% gave qualified approval in case an additional year could be had; about 13% declared emphatically that the engineering colleges undertake too much; that there is too much stuffing with facts and details and not enough thorough work on basal principles. Scattering criticisms were to the effect that mechanics is slighted and literary training and attention to elementary forms of business procedure are too much neglected; that there is lack of completeness and mastery even of many things like drafting, note-keeping, etc., in which the graduate professes to be adept. About 70% had never known a young graduate equal to considerable responsibility; one said yes, and the others no, excepting cases where previous experience or special training had been enjoyed.

Even the most diverse of these opinions are probably nearly correct; for contradiction is justified by the confessed differences in the qualities, capacities and attainments of the graduates. It is a matter of common observation that young men of the cities, nurtured in comparative luxury, living at home during their student days, who have been so unfortunate as to know but little of struggle

and hardship in early years, usually lack self-reliance, energy and power of initiative to a marked degree. The sturdy characters made by some hard experience in helping to earn the family living, either on the farm or in the shop, who have gained practical sense and habits of thrift, are generally the most promising material for the engineering school to deal with. Again, certain localities, especially where the habitat of the engineering college is in or near a large city, abound in a superfluity of graduates, loth to leave the familiar environment, who are willing to work for the lowest living wages, in anything that they consider to be in the professional line. Here the argument that such a preparation is suitable for other and related pursuits is true indeed, but only in part; there are always some incompetents who finally drop out because "half baked," misfitted, or otherwise disqualified.

Just here we may notice some examples of education continued beyond the college period—a sort of reversion to the old apprenticeship (and yet not a reversion, really, because the first years of practice must ever be an apprenticeship). Certain of our largest corporations which control many lines of industry have successfully adopted the policy of taking young men, preferably technical graduates, into their various shops and offices for a period of one, two or three years of special training and instruction in particular branches.* This realizes the true idea of the school for the specialist at the right time. Here is close contact with latest and best practice, no lost motion, and effectual training. In at least one instance the manager prefers to put the young men into the shop at first, to learn by hard work and close contact all the routine of handling and assembling the material (this is in structural work of miscellaneous sort), so as to become assistant foremen and inspectors; then, after about a year and a half, to promote them to the offices where the intimate knowledge of details just acquired fits them to master rapidly the technique of designing and esti-

* To show one instance the following is from the report of a Director in charge of a visiting party of engineering students: "We were informed that the General Electric Company employs about three hundred and fifty collegians, or trained technical men, who come from all parts of the United States and from abroad. A high-caste Hindoo and bright-looking Chinese student were pointed out, and a due proportion of Japanese are not wanting. These young men are employed as "testers," inspectors and assistant engineers, beginning commonly with pay of less than six dollars per week. Yet the places are much sought for because the really capable ones are gradually promoted, and an apprenticeship of two or three years here gives great prestige to one who seeks wider opportunities beyond."

mating.* This process produces a true expert in about three years, with progressive increase of pay.

Some railroad corporations have adopted a policy of employing young technical graduates to build up an engineering staff, but in certain cases, owing to change of management or plans, have failed to fulfill the promise of steady advancement which induced the men to devote some of the best years of life to the purpose of becoming experts in railroad engineering.

The individualism of the various institutions—determined by past history, achievement, environment and acquired momentum—will persist. It is an inevitable condition. It accords with the national spirit. Amidst this necessary diversity there is one principle of unity and common aim which determines a common ideal of success. Efficiency is the great end sought in all engineering operations—getting the largest results possible from a given outlay. The engineering college or institute must be brought to this test.

The self-evident elements of such efficiency in this relation are: The quality of the student material; the personal equation of instructors as to character and competency; the right selection and co-ordination of the subjects taught; the methods of teaching and applying tests; the policy of administration in relation to discipline and management of the cost. Finally, regulation of the intensity of the process is essential. The old-time school, dealing with a narrow range of work, could be deliberate; now the body of science and practice is so great that there is a tendency both to undue haste and to overloading the system. The rate of working must be properly adjusted to the resistance.

This ideal, therefore, demands respectively: Rigid test of the aptitude, preparation and character of the student at entrance; thoroughness of instruction during the course—quality rather than quantity; and strict conformity to a high standard of graduation, made as uniform as possible throughout the country. The right qualities in the personnel of the teaching force are so obvious as to need no statement here. As to the subjects taught, the fundamentals cannot be omitted, although they are slighted too much in all departments of education. Efficiency demands first a sufficient foundation—of all primal definitions, principles and methods—on

* U. S. Steel Co., North Chicago Works.

which any chosen professional career may be readily and surely built; also such co-ordination of all the subjects that each shall have due proportion according to its real educational value; also a sufficient number of so-called "culture" studies to keep the balance and educate the whole man. This co-ordination depends upon concurrent and harmonious action of the entire teaching force; hence it is not easily attained, and the lack of it leads to much ineffective work—by the overdoing in less important subjects, the slighting of the more important, and the creating of a false perspective for the student. The methods of teaching must be dictated by conforming to the principle that the student learns thoroughly only by his own hard work; that there should be much more suggestion and direction than assistance. Whether by textbook, lecture, or practice in laboratory, field or shop, instruction is the end sought. Overdoing by any one of these agencies results in waste. Discipline in the engineering school ought to be automatic; since a prime qualification for the engineer is strict attention to duty, obedience to his superiors, and an intuitive sense of responsibility, the student who fails in these respects rules himself out as incapable. Efficiency as affected by cost has been much impaired in well-known instances by needless expenditure on buildings, to the detriment of the endowment funds, and while the instructors were yet underpaid. Correct policy makes the cost of men primary and seeks the most competent at any reasonable expense, while it holds as secondary (although much more indispensable than formerly), buildings and all forms of material equipment; adjusting the outlay therefor strictly to actual needs. As to rate of working, the intensity so common in some quarters is harmful. The forced draft is not economical, either for men or engines. The shorter and well-chosen programme well done is far more effective than the forced and hurried work on longer courses, with the too common results of broken health and broken hopes.

There is another element of efficiency which almost eludes statement and of which no suggestion appears in any programme. What shall make sure the personal integrity and honesty of the graduate? These are days of special temptation; the bribing of inspectors has long been notorious; the engineering profession is in constant touch with political corruption everywhere; the engi-

neer's ideal of the best possible material and workmanship at the least cost to the public or individual is directly contrary to that of many municipal managers and contractors, whose attitude is: "What is there in it for us?" The engineering college or institute which fails to set the highest moral standard before its students is seriously delinquent. The most notable engineers of the past have been distinguished for a delicate sense of honor, which shunned even a remote connection of personal interest with professional duty. Such men were above suspicion under trying circumstances. It should be part of the education of the student to become acquainted with the history of his profession and especially with the biographies of its honored exponents. And not only the past but the present will furnish him practical ideals of duty and integrity among the leaders of the profession to-day.

In conclusion it appears that, during the past decade, there has been constant improvement in the adjustment between the engineering colleges and the profession at large. While the "practicians" must always lead in the development of the practice, the "schoolmen" are promptly striking the balance, and giving to the profession the textbooks or treatises which embody all the latest essentials of the science and the art. Moreover, the successful operation of laboratories of research at the colleges (*e. g.*, the hydraulic laboratories, laboratories for testing materials, locomotives, etc.), have set an example which corporations and individuals have not been slow to follow; and these are really more closely related to the profession than to the legitimate work of instruction. Here is where the old distinction between schoolman and practician disappears, for they meet on common ground to interrogate Nature. Endowment of research in engineering is proposed in some universities and institutes, and is already accomplished in the Carnegie Institution. In examinations, under law, for civil engineers in the United States Navy, one requirement is a diploma from an engineering college. And in other cases the same qualification is demanded. But the value of such a diploma depends upon its source. It is to be hoped that within another decade, testimonials from all the leading engineering colleges will approach more nearly to an equality in value.

The demand for technical graduates is greater than ever before.

At the present time the supply exceeds the demand; and this situation is likely to become more acute, because now in the leading colleges the tendency of the students is to turn from the old classical and humanitarian learning and seek the scientific and utilitarian, which promises more immediate entrance into the useful activities of life. It is evident, therefore, that a great responsibility rests upon those who are charged with the directing of engineering education. Endeavor should be made for concurrent action directed to these ends: Careful selection of the student material at entrance so as to exclude rigidly the unfit and unworthy; the incapable should be mercilessly weeded out in the process of training; the final tests at graduation should be so impartially applied that the testimonial of the institution should everywhere command confidence. This would result in a healthy restriction of the number and a much needed improvement of the quality of the technical graduate. Consequently, the common purpose persistently directed to the realization in all respects of the proposed ideal of efficiency, would secure a high degree of excellence and uniformity in the value of the output.

TRANSACTIONS

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INTERNATIONAL ENGINEERING CONGRESS,

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Paper No. 17.

ENGINEERING EDUCATION.

By Calvin M. Woodward, A. B., Ph. D.

Engineering Education is a very comprehensive term, yet it has, or should have, a clearly defined meaning. This paper is written for the purpose of defining Engineering Education in its broadest sense and of pointing out its more recent developments and characteristics.

While formal educations of more or less definite types have existed for many centuries, Engineering Education has but just taken shape. Engineering itself is the product of the last fifty years, being the flower and fruit of the great discoveries in the physical sciences.

Engineering is both a science and an art, and Engineering Education concerns itself with those scientific principles which underlie engineering arts, and with as many of the arts themselves as illustrate clearly fundamental physical laws and exhibit typical methods of making them serviceable.

This last word is the watchword of Engineering Education. It is to be serviceable. The idea of service underlies every detail of it, and that service is objective, altruistic, and therein it differs from that older education whose supreme object is "culture," which, as Emerson says, is valued not for what it enables one to

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accomplish for others or for the world, but for what it is supposed to accomplish in the student himself.

The writer has the greatest respect for the noble seer of Concord, and feels sure that were he living to-day and fairly in touch with the content of Engineering Education, he would agree in the assertion that true culture cannot exist apart from a conscious preparation to be of service to others. No man lives nobly who lives for himself alone; and, on the other hand, no man can acquire an education which will enable him to be of eminent service in the world's work, without experiencing in his own life and character that refinement of taste and that love for what is best and truest in word and deed, which always mark the cultivated man. The accomplished engineer is perforce a cultivated man, though as a rule he is the last to claim consideration as such. The poem of "Abou-ben-Adhem" should be rewritten. It should read:

I pray thee then
Write me as one who serves his fellow men.
And on the shining scroll his name will still "Lead all the rest."

But the writer claims for Engineering Education no monopoly of high aims, or of men of service. History is full of the records of men who, like Emerson, did splendid service for humanity, though they knew little or nothing of what is characteristic of the modern engineering curriculum. We all know that there is more than one avenue to culture. In point of fact there are many avenues, and one purpose of this paper is to claim for the accomplished engineer his right to equal and full membership in the increasing brotherhood of culture.

And yet men have frequently deplored the fact that certain studies have been found useful. It has been difficult for the writer to accept the evidence of his own eyes when he has read of the regret of college professors that certain college studies have been found useful in engineering. It is hard to accept this statement, but it is true. Recently the writer called public attention to the words of a professor, of one of the most renowned universities in the United States, who said:

"With the increase in the utility of studies, they will be of less value in educating men. Whatever makes them more fitted for utility studies, makes them less fitted for general culture."

Hence mathematics and electricity have less educational value than formerly when they were of less utility. Again he asked:

"Shall a subject be taught with a view of making a utility from it? or shall it be taught with the purpose of producing the greatest effect upon the mind and culture of the pupil?"

The experienced teacher will not fail to see a fine distinction here. The seeker after culture must be careful not to learn his subject thoroughly. If he go too far, and master a subject, and make it good for something, he will miss the object of his search! Instead of being a "man" or a person of general culture, he will be only an "expert," or an "adroit" engineer.

But Professor Patton uttered that superb foolishness thirteen years ago. Doubtless he became wiser later on. The young men in American schools of engineering in this twentieth century are not given such notions. They are taught to master subjects, as fast and as far as possible, so that they may be of use to them in the economy of life, or as a means to continued study in the mastery of other subjects. It is only the snob who has a dread of learning something useful. The educational army has changed front, and the prejudices against the utilities are fast disappearing.

It is easy to understand the source of the old prejudice against technical training. The history of civilization has been the history of masters and slaves, of caste, of contempt for labor and for all useful arts.

Every one of the technical professions had its beginnings in the crafts, and the present expert and chief engineer had as a prototype a man in overalls, with horny hands and an eager face, who presided over some enginery which was not in the books, and who was regarded as ungenteel.

Milton placed Memnon, the first ante-tellurian engineer, among the fallen angels and sent him

"With his industrious crew to build in hell."

The education of an engineer not only aims at a mastery of fundamental principles and a familiarity with their applications in the constructive arts of our modern life, but it possesses the splendid property of arousing a lively interest in the student and of stimulating his intellectual energy, and often his physical energy, to the

highest pitch, so that study and laboratory practice are both pleasurable. And here again the old prejudice breaks out in a new form. Recently, in the *North American Review,* the writer ran against an advocate of uninteresting studies for the purposes of mental culture. The writer has no space to review Professor Wendell's article, which has many good points which he heartily endorses, but he must quote three sentences which it is safe to say no engineer and no teacher of engineers would ever have written. He seems in some degree to share Professor Patton's fear of useful studies. He says:

"The practical aim of a general education is such training as shall enable a man to devote his faculties intently to matters which of themselves do not interest him."

"The very fact that the abstractions of mathematics must generally seem repellently lifeless, is part of the secret of their educational value."

He says further that one reason why the natural sciences do not have greater educational value is because they "are apt, nowadays, to prove a shade too interesting."

If the writer were to discuss the doctrine of "interest," he would be carried far astray from his proper theme; yet it is not altogether foreign to those engaged in the education of engineers to call attention to the fact that a Harvard professor, though speaking only for himself, makes such statements and takes a position which must carry him to such results as these:

1.—That it is harmful, educationally, for a student to become deeply interested in his studies; and,

2.—That the most efficient method of training a man, presumably to be of some use in affairs, is to confine his attention during all his youthful years upon matters in which he takes no interest, and "which of themselves could not hold his attention for five minutes together."

The writer is confident that every engineer will agree with him that the deeper one's interest in a study, the better in every way, educationally and otherwise, so far as that study is concerned. It is true all studies are not equally interesting, and it may be harmful to allow a lively interest in one subject to draw one away from subjects which are less inviting. But the idea that it can possibly

be better, morally or mentally, for a student to find his studies uninteresting than exceedingly interesting, is, the writer feels sure, obnoxious to every one who knows anything about Engineering Education. The zeal of engineering students is proverbial, and not one of us wishes it to be otherwise.

Our friends from over the Atlantic will see at once how foot loose we are. We do not hesitate to challenge the traditions, to pull down the idols our fathers worshipped, and to prefer rationalism to dogmatism at every point. We do not scorn precedents, but we are not greatly hampered by them. It is the privilege, though not always the duty, of the able engineer to make precedents. The foundations and the ribbed arches of the Eads Bridge over the Mississippi in St. Louis, the great cantilever over the Firth of Forth, and the Eiffel Tower surpassed all precedents, but ignored none of them. A thorough knowledge of the laws of mechanics and an intimate acquaintance with the nature of materials sufficed for new creations. The accomplished engineer is a creator, and he feels himself such. This feeling is one of the rich rewards of the profession. It reconciles one to the shrug of the traditionalist and the disdain of the scholar. The engineer feels secure in the essential nobility and dignity of his profession, and in the service he renders to mankind. More and more is the world coming his way, and more and more is the general culture value of technical training appreciated and sought.

The writer has said that the educational army was changing front. Statistics show a vast and rapid increase in the number of technical schools of all grades within the past ten years. Professor Fletcher in his paper has given the facts more fully than they have been given elsewhere. His report leads to the conclusion that in another decade engineering education in all its various forms will be the most popular kind of higher education for young American men.

It will be observed that Engineering Education not only affords professional training, but is a form of general training into which may enter no definite thought of a professional career. Modern life is so closely related to engineering operations and engineering results, that a thorough training in the fundamental principles of mechanics, and an extensive laboratory experience form the best possible basis

for success in life in the highest sense—success measured by the service one can render to others, and the happiness which the consciousness of rendering service always confers. More and more is it seen that success in occupations which at first glance appear remote from engineering, after all depends upon adjuncts and appliances which are strictly the province of the engineer. Questions touching the generation, transmission and utilization of mechanical power are business questions as well as engineering questions. If one wishes to see how engineering matters are mixed with commercial, let him visit "Cupples Station" in St. Louis, where at a single platform, by judicious grouping and the freest use of appliances, all essentially engineering in their nature, over 50 cars with 1 000 tons of assorted merchandise are received and shipped every day.

Professor Fletcher says that at the present time the supply of technical graduates exceeds the demand, although 80% of his correspondents think otherwise. The writer fancies that one's opinion on that matter depends greatly upon his point of view. He is one of the 80 per cent. He thinks that the supply in the City of St. Louis, at least, is less than the demand, much less, even when reference is had to strictly engineering positions. And when we take into account that young engineers, once in the service of industrial, transportation and commercial concerns, are fast rising to responsible positions as superintendents and general managers, it is easy to see that the supply is greatly inferior to the demand. In the writer's judgment, the trend of higher non-professional education is away from the traditional means and methods, and toward the modern means and methods which are grouped very largely under the banner of Engineering Education. Professor Fletcher has with rare skill, from his storehouse of exact information, given us the history of American engineering institutions up to date. The writer hopes he will live to write another chapter, ten years from now, and may all of us be present to hear him read it.

Among the developments of the last ten years, Professor Fletcher referred to the Society for the Promotion of Engineering Education. The writer wishes again to commend that Society and its annual reports to favorable consideration. In proportion to the interest in the education of an engineer will be the interest in the work of that Society. The topics which have been discussed at its

meetings need not be enumerated, but attention is called especially to such as these: "Entrance Requirements"; "The Evils of Too Great and Too Early Differentiation of Courses of Study"; "The Proper Balance between the Class-Room and the Engineering Laboratory"; "The Abuse of the Lecture Method of Teaching"; "The Folly of Large Sections in Subjects Involving Mathematics, Physics, and Mechanics"; and "The Confusion of Degrees."

Americans should join the Society; and those who live beyond our borders should send annually for its reports.

Reference has been made to the rise and development of manual training, whereby Engineering Education has been better planted and more successfully cultivated. The writer's relation to manual training as a feature of secondary education has been such as to entitle him to speak of it with confidence if not with authority.

Thoughtful men in all lands during the past 300 years have insisted upon the general value of a craft thoroughly learned. It was seen to be wholesome both mentally and physically for a boy to learn a trade, and writers like Newton, Rousseau and Pestalozzi maintained that contact with things and material forces was morally helpful as well. But no one seems to have thought it possible to get the moral and intellectual benefit of trade culture without actually learning a trade, and serving an apprenticeship in the old traditional way; and as an old-fashioned apprenticeship involved an abandonment of the school and other forms of culture, nothing came of such views and convictions. No one cared to have his boy learn a trade if the price he had to pay for it was from three to six of the best years of the student's life. Nevertheless, with the development of the physical laboratory, and the introduction of experiments to illustrate the principles and determine the constants of engineering, there grew up a conviction that somehow students in engineering should know the theory and be familiar with the use of typical hand tools for working the ordinary materials of construction. So George Whitworth (later Sir George) of England, sent engineering students into an ordinary shop; and Professor C. O. Thompson, of Worcester, Mass., organized a commercial machine shop and put his engineering students into it, so that while being trained for engineering they might become practical machinists.

But, best of all, a Russian director of a government engineer-

ing school, Victor Della-Vos by name, conceived the plan of teaching the theory of tools and the typical ways of using them, in a shop especially fitted up for the purpose of instruction alone, and then sending them into a construction shop at a later stage. Thus he separated the instruction or pedagogic period from the construction or commercial period. To be sure his youngest pupils were eighteen years old, and the two stages before mentioned covered six years in all, while the young men were taking the ordinary studies of mechanical engineering. This plan was put into successful operation in 1868 by Professor Della-Vos, of Moscow, and, in 1876, he exhibited his theory and practice in the Centennial Exhibition at Philadelphia. The writer saw the exhibit and so did Professor Runkle of the Massachusetts Institute of Technology, and the latter made an elaborate and very valuable report upon it. The writer had already had, for three years, a crude instruction shop for teaching young engineers the use of tools, and while Professor Runkle organized a section of his engineering students in metal-work on the Russian plan, the writer tried an experiment of elementary woodwork with a class of students of secondary grade. In 1877 three shops were fitted up in Washington University in St. Louis, in an old dwelling-house, for classes in woodwork, forging, and machine-work. A report upon this work, and upon that of Professor Runkle in Boston, published in 1878, gave rise to the St. Louis Manual Training School in 1879. The money to establish, enlarge, maintain and endow this last school was for the most part given by eight men, three of whom are still living. Such, in brief, was the origin of the manual training school, a secondary school, with a liberal course of study in mathematics, science and language. It incorporated, as essential and co-ordinate features, regular class exercises in toolwork of several kinds, and regular exercises in the rudiments of mechanical and free-hand drawing. Its daily programme was simple. The day consisted of six hours. One hour was invariably given to mathematics, one to science, one to literature and language, and one to drawing, and the other two to the theory and use of tools. This programme covered three years. The toolwork for all included joinery, carving, turning in wood, pattern making, moulding, forging, bench and machine-work in metals.

This was a school with an original programme. It bore little resemblance to a trade school, of which there were at the time none in America, though there were hundreds in Europe; and it did not follow either the organization, course of study, or grade of the Imperial Technical School of Moscow. What it owed to Della-Vos was the scheme for teaching tool-theory apart from ordinary shop practice, and his method of mechanical analysis. This obligation is cheerfully acknowledged.

Professor Fletcher has written of the rapid spread of manual training in America. Had he cared to go beyond America's borders, he could have told of manual training schools all over the world, organized very closely on the St. Louis plan. The writer has reports of such schools from Australia, from Hawaii, from Gordon College on the Nile, from the Barbary States, and from islands in the Indian Ocean.

The writer does not hesitate to call attention to what has been done and is now being done by the Board of Education of the City of St. Louis, in incorporating manual training in the public school curriculum. St. Louis may have been slow in taking up manual training in its public schools, but it is not lacking in present zeal and energy. Every boy in the seventh and eighth grades of the city grammar schools is given a two-hour lesson per week in elementary toolwork in wood; and advanced woodwork and metal-work is offered to all the boys in two large high schools. It may be said, in parenthesis, though it is only remotely connected with Engineering Education, that all the girls in the same grammar grades are given elementary domestic science; and, in the high schools, advanced work in the household arts and in the fine arts is offered to all.

In the new high schools of St. Louis, it is thought that the high-water mark has been reached as regards equipment and logical teaching in all the departments of instruction. Each building is fitted for 1 000 pupils, boys and girls, and each represents a total expenditure by the Board of Education of $500 000. Public education is free and, in St. Louis, textbooks, drawing instruments, etc., are supplied free. Both these high schools were opened in 1904, one in February and the other in September, and they bid fair to increase the high school attendance of the city by 2 000 by September, 1905.

There is abundant evidence of the fruit of manual training in the careers of the graduates of the school whose origin has been described above, and which is under the charter and control of Washington University. Of the 1 000 graduates of that school over 200 have entered schools of engineering. It has thus fulfilled one of the great objects for which it was founded. For years this statement has been published in the annual catalogue:

The St. Louis Manual Training School "was organized to effect several ends:

"1.—To furnish a broader and more appropriate foundation for higher technical education.

"2.—To serve as a developing school where pupils could discover their inborn capacities and aptitudes, whether in the direction of literature, science, engineering, or the practical arts.

"3.—To furnish to those who look forward to industrial life opportunity to become familiar with tools, materials, the methods of construction, and exact drawing, as well as with mathematics, elementary science, and ordinary English branches."

It would be difficult to say in which of these three directions the school has been most successful. The high rank and professional success of those graduates who have become engineers are well known. Its work as a developing school is less noticeable, for the reason that a boy's development makes no noise, and always seems natural and normal, and statistics on this point are not available. But the directions in which its graduates have moved, and their efficiency along lines of activity evidently well chosen, show that the all-round programme of the school has had the effect of arousing innate aptitudes and of opening up attractive fields of labor. The table is a record of their occupations a year ago.

Occupations of the St. Louis Manual Training School Graduates: Classes, 1883-1903.

Agriculture	14	Chemists	9
Architects	24	Contractors	2
Artists	4	Dentists	4
Bankers	7	Draftsmen	100
Bookkeepers and Clerks	153	Electricians	19
Cashiers	5	Fieldmen	4

Foremen	3	Real Estate Men	18
General Managers	32	Reporters	2
Insurance Men	9	Salesmen and Agents	41
Lawyers	30	Students	75
Librarians	1	Superintendents	44
Mechanics	14	Teachers	39
Merchants and Manufacturers	90	Technical Engineers	65
Ministers	1	U. S. Navy Engineers	4
Physicians	22	Miscellaneous	15
		Unknown	56

Of these, 159 have taken degrees in higher institutions of learning.

This table was prepared as an answer to the frequent question: What becomes of your graduates? There was a strong feeling in many quarters that they would not turn out well, either professionally or socially. Plato had taught that the practical arts were degrading; and he had and has many disciples. The belief was common that polite learning, so-called, made polite men, and that contact with materials and physical forces made degraded men, but does it? One anxious mother said, "If you teach my son carpentry, he will seek the companionship of carpenters." Possibly he may, to a certain extent. He might do worse. A Philadelphia paper predicted that the Central Manual Training High School of that city would "turn out a mass of degraded operatives." In point of fact the graduates of that excellent school show a strong desire and tendency to enter the University of Pennsylvania. While the above table fails to prove conclusively that the young men of the St. Louis Manual Training School chose wisely and well their occupations in life, and that their contact with the practical arts, so far from being degrading, was humane and liberal—yet it does establish a strong presumption that manual training is not only no bar to professional and business success, but that it gives a signal advantage.

This result was not fully anticipated, and yet it constitutes the choicest fruit of manual training, and in this respect it resembles the fruit of all sound engineering education, and for that reason it is worthy of a moment's consideration.

Tool instruction when properly given is associated closely with a careful analysis of the processes of construction, so that what at first may seem a complex construction or operation is found to consist of a series of simple steps, which are shown to fall into logical order. A construction is no longer difficult when its parts are seen to be easy. A process is no longer complicated when the steps are seen to be simple, and their proper order is found. Now any one can see the sure mental and moral fruit of such training. It is not limited to the shop. The analytical habit of mind once acquired is valuable in every department of study, in every occupation in life. It is no wonder that young mechanics (when they have a strong natural bent for mechanics) soon become foremen, and superintendents, and general managers. Having acquired the art and the habit of thoughtful self-direction, they easily become the directors of other men.

One of the graduates chanced to meet the writer a few years after leaving school, while he was still a young man. The writer asked him what he was doing and how he fared. Said he: "I am a blacksmith, and I am doing well." The writer was at once interested, for though every boy in the school learns the alphabet of the forge and constructs finally for his own use a complete set of lathe tools, no man of them ever had reported himself as a blacksmith. So this young man was asked to give particulars. Said he:

"My father was a blacksmith, and when I left school I went into the shop to help him. But I found his shop very incomplete. It lacked tools, and he could not handle heavy work. So I set out to enlarge and improve the shop. I put in new tools and power. Now I have things fixed in good shape. I have twenty-six men under me and am prepared to do forging of any size or character. My father has retired, and takes life easily, and I am running the business."

The writer regards that young man as a type of hundreds. His school education stopped with the manual training school. Had he gone on and taken a full course of engineering, he would no doubt, in a still more remarkable manner, have shown in his life and character the legitimate fruits of a sound and thorough education extending through both the secondary and higher periods.

Manual training is fast doubling the high school attendance of

boys, and it does this without interfering to any great extent with the attendance at literary schools, and this is just as it should be. When the writer was a boy it was felt and plainly said, if a boy does not care for Latin, Greek, and mathematics, as then taught, and if he is not likely to be sent to college, then let him go to work; there is no school for him. So some of the brainiest heads, and a large proportion of the boys born to be leaders and men of action, got little formal education beyond the common grammar school.

This new century is training such boys in high schools by the thousands, and our colleges and schools of engineering are being taxed to their utmost to provide adequately for the graduates of the high schools.

The problems of Engineering Education are not all solved, but, with the help of the engineers themselves, they are in a fair way to be solved.

TRANSACTIONS
AMERICAN SOCIETY OF CIVIL ENGINEERS.

INTERNATIONAL ENGINEERING CONGRESS,

1904.

DISCUSSION ON

ENGINEERING EDUCATION.

By Messrs. J. L. Van Ornum, D. H. Ray, Frank O. Marvin, H. N. Ogden, Alfred Chatterton, Sir William H. White, Robert Fletcher and Calvin M. Woodward.

J. L. Van Ornum, M. Am. Soc. C. E., St. Louis, Mo.—In considering the work of engineering colleges, which comprises really all education that directly concerns the professional engineer or the educator except in so far as he is interested in adequate preparation for the engineering college, the speaker desires to emphasize one essential factor of Engineering Education, that seems almost submerged in the aggregate of valuable facts and opinions expressed by Professor Fletcher. The adequacy of the instruction, both in its substance and in its spirit, is the vital point in the consideration of the question. This brings at once into the foreground the qualifications of those responsible for this education; qualifications which must be such that the knowledge imparted may be theoretically correct and practically applicable to the varied needs of the profession, and that education shall not end with instruction only, but rather that the foundation may be laid for straight reasoning, systematic effort, personal integrity and independent thought. Mr. Van Ornum.

To give instruction theoretically broad and sound implies that the instructor must be one whose own training has been thorough and broad, and who is able to impart successfully his knowledge; to make this instruction also professionally useful requires a man who has had professional experience himself, and who still keeps in touch with the rapidly expanding requirements of the engineering

Mr. Van Ornum. profession. Nor is this all, or the intrinsically greater part. In order that scholastic instruction may develop into a veritable education, it is necessary that the spirit of the work must increasingly develop in the student the talent of self-reliant judgment and of devotion to the truth which the scientific work of the laboratory and the classroom fosters, but which may be vastly reinforced by the personality of the educator. He must not be a man of vacillating enthusiasms; nor one who uses his position as a stepping-stone to another; nor one of narrow sympathies or of negative character; nor one displaying a suspicion of uncertain power or of doubtful attainments. And if such a combination of attributes seems impossible, let us remember the story of the late Charles S. Storrow, Hon. M. Am. Soc. C. E., mentioned by Professor Fletcher in the first part of his paper, and recall the names of some whose administration of college departments has illustrated these truths strikingly.

The speaker believes that the personality of the educator is the kernel and essence which permeates profoundly the whole field; that scholarly attainments, engineering experience, rigid integrity, inspiring idealization, broad humanity, zealous activity, sympathetic insight, and devotion to his calling, in so far as they mark his character, signalize the progress and promise the success of Engineering Education.

Mr. Ray. D. H. Ray, Jun. Am. Soc. C. E., New York City.—Two ideas have been placed before us for discussion. They might be indexed under the general topic of Engineering Education, as (1) culture; and (2) manual training. The latter has been treated at length, but not much has been said about the former. To define culture is not easy, but it calls to mind a conception of refinement acquired by mental and moral training as opposed to purely manual training. It implies an understanding of what is meant by civilization. It implies that broad outlook upon affairs which is attained by carefully directed study continued along many lines of mental endeavor, and it demands that refinement in manner and taste which defines the word, gentleman. Culture is in part the true end of education, and it may include, as a minor subordinate means to that end, the training called manual training.

If we substitute for the broad term, culture, that branch of scholastic training by which it is most often sought to be attained, *viz.*, classical training, we have for discussion two ideas in preparatory Engineering Education: (1) classical training and (2) manual training.

The first implies more or less of the course of studies which leads to the degree of Bachelor of Arts. The second implies the common branches, with a predominance of manual and shop training. They are not incompatible. To make either an end, rather than a means

to an end, serves no useful purpose. Manual training may be administered just as foolishly as classical training, and even to less purpose. A smart boy with a bent for engineering may learn more in half an hour by watching a skilful mechanic than a manual training school will teach him in a week. He may be just as much disheartened and discouraged by the manual training hour as by the Latin hour. The manual training system was imposed on the elementary schools of New York City some years ago. The fad distracted both teachers and pupils. Boys were taught sewing before they had mastered reading, writing and arithmetic. Mr Ray.

Either a classical or a manual training may be a success or a failure, as the foundation of Engineering Education, according to circumstances. Both systems together may be conceived to fail. The ideal system is neither the one nor the other. It is, in fact, no hard and fast system, but rather a course of procedure designed to develop, by tutoring, each particular student to his highest capabilities. This leads us back to the derivation of the word, education. Education means the action of leading forth, bringing out, developing morally and intellectually.

Manual training alone cannot do this. Classical training has been successfully applied to this end in the past. It would seem, therefore, that for the highest type of engineering schooling, the classical studies are desirable as a foundation. Such training has a legitimate and a real value to those who seek those qualities of mind and heart which are just as necessary as technical ability in the administration of large engineering work. Manual training has a broad and proper field in developing handy, skilful subordinates. The Chief Engineer of the New York Subway can sign A. B. before his C. E. Some of the efficient assistants and foremen on that work are manual training graduates.

The speaker has given part of his time for five or six years as an instructor in elementary engineering in New York City. The facts that impressed him most during that time were:

1.—The foolishness and the positive harm done by educational fads, such as manual training.

2.—The necessity for careful study by instructors of the aptitudes and aspirations of their students, not only that they may be well taught, but that they may be advised and encouraged to make the most of themselves in the direction of culture.

3.—The apparent decrease in efficiency of the efforts of instructors after five years of teaching in single, limited subjects, due to lack of sustained interest, or too great proficiency, or falling behind.

4.—That both classical and manual training are useful as a foundation for technical education, and that the choice depends on the aptitude, energy and circumstances of the student.

Mr. Ray. If the full classical course precedes the technical course, the latter becomes a graduate course. The student then takes up his life work later; he is more mature, more capable of making a decision and better prepared by previous mental discipline to attain success.

In France, England and Germany, it is believed the students study for a longer period than in America. They have the incentive of government employment which no American schools, except the Naval Academy at Annapolis and the Military Academy at West Point, enjoy. This must encourage graduate work in such schools as the École des Ponts et Chaussées.

Mr. Marvin. Frank O. Marvin, M. Am. Soc. C. E., Lawrence, Kans.—A point of some interest in this discussion, which might be mentioned, is the influence that Engineering Education has exerted upon the teaching of subjects forming parts of the older curricula. Take mathematics, for example: Engineering instructors have often found that its teaching had been handled from so abstract a point of view that they were obliged to do some of the work over again in order to connect its branches, the calculus, or even the trigonometry, as these had been learned, with their applications to engineering studies. The demands of instructors in applied science, coupled with the fact that mathematical teachers themselves are finding the old methods of teaching their subjects as "pure" unsatisfactory in results, underlie quite a strong movement among the mathematicians in the United States, which looks toward enforcing principles by showing at each step their concrete applications, and their relations to the practical facts of Nature. This change in point of view and method is designed to affect all students, those in general as well as in engineering courses.

The trend is further illustrated by the marked change in the character of textbooks. Compare, for instance, the latest book on mechanics, Maurer's "Technical Mechanics," not called theoretical though sound in theory, with those used twenty-five or thirty years ago.

Dr. Woodward in his admirable paper touched a point that cannot be too strongly emphasized, and that is the fitness of an Engineering Education as a training for a business life. The directness of aim, the concentration of personal power to attain a given end, the dealing with the facts of Nature and correlating them, the training in observation of phenomena, the weighing of the relative importance of facts observed and the drawing of logical conclusions from them, all these lead a young man to recognize the correct methods of analysis and the true methods of thinking. Further, they lead him to a respect for his conclusions, to a belief in the truth of his results to a much greater degree than would be given him by the older forms of education.

In the speaker's opinion, these and other qualities developed by an engineering training are just those qualities which are needed in business life in the industrial world, so that this training is better than any other existing form of education in preparation for that life. Yet, as many young men who enter engineering schools are better fitted by their natural qualities, as well as bent, for the executive side of a business career than they are for professional practice, it becomes a question of considerable importance whether our technical schools should not more fully consider the needs of this class than they do at present. Mr. Marvin.

As to the matter of engineering research, the schools of America have been too busy in their rapid growth and development, and in trying to meet the demand for their product, to do much systematic work in scientific investigation. That there is a field for such work, as a part of an educational policy, there is no question. That it will be occupied, and the money therefor be provided, both by private gift and through State aid, seems also certain. The trend is evidenced by the establishment of schools of research at the Massachusetts Institute of Technology and the University of Illinois, the opportunities for the study of hydraulic problems afforded at Cornell University, and the reports of special investigations coming with greater frequency from various colleges and universities.

Mr. Ogden. H. N. Ogden, Assoc. M. Am. Soc. C. E., Ithaca, N. Y.—It has been very interesting and entertaining to hear the quotations in the paper of Professor Woodward, but it is probable that few engineers will be found to acquiesce in the extreme opinions, as regards the value of humanitarian studies, advanced by Professor Wendell. Professor Woodward, in presenting evidence on the other side and arguing in favor of the cultural value of purely technical subjects, permits an inference which, it seems to the speaker, is hardly true. It is not uncommon to see, in a technical school, men, whose home associations have been lacking in refinement, whose vocabularies are limited, whose knowledge of life is narrow and whose outlook is restricted, gradually develop strong mental powers. Such men become able mathematicians, capable engineers and thorough scientists; but that such men, without more liberal training and with no common ground on which to meet other men, even with the highest ability in their specialized field, can be called cultured is an inference possibly to be drawn from the paper, but with which the speaker has no sympathy. Probably the author does not intend to approve of the extreme conditions here assumed, nor mean to suggest that technical instruction in itself can make up for deficiencies in general education, but the speaker desires to emphasize the special necessity of technical men not neglecting other and broader reading and study.

Mr. Chatterton. ALFRED CHATTERTON, ASSOC. M. INST. C. E., Madras, India.*—The speaker's practical experience in Engineering Education has been confined to India. There they have a peculiarly difficult class of students to deal with. Most of them are Brahmins with very retentive memories and considerable intellectual capacity, but with defective powers of observation and an entire absence of manual skill or handiness. The Government of Madras was persuaded to provide suitable workshops and laboratories, and the students underwent a course of manual training, during the whole of their college course, of the type advocated by Professor Woodward. Personally the speaker was extremely satisfied with the results, and so were the officers of the Public Works Department, under whom the majority of the students subsequently served. For a number of years the introduction of manual training into the colleges and high schools of India has been advocated, but the country is still too poor to afford such an expensive system of education. Primary education occupies a good deal of the attention of the educational authorities at the present time, and a much more rational and practical system has been introduced lately which it is hoped will be gradually extended upward, till it culminates in schools of the type of the St. Louis Manual Training School.

The speaker has visited a number of engineering colleges in England, Canada and the United States, and whilst admiring the thoroughly practical way in which most of them are equipped, he has been struck with the fact that there seems to be a tendency to over-elaborateness in the nature of the experimental apparatus and machinery which has been set up. Experimental steam engines figure largely in the modern engineering schools, and it is very doubtful if students obtain from them quite as much useful practical knowledge as their designers seem to have expected. They are too complicated and too far removed from the everyday engines of the workshops to have much practical value. Tests on such engines require too many observers, and the working out of the results occupies an amount of time disproportionate to the value of the instruction obtained.

The idea that students in engineering colleges can be employed on research work with profit to themselves or any one else is absurd. An engineer cannot be trained in a college, or a college laboratory, and the aim of such institutions should be confined to instruction in the scientific principles underlying the practices of the profession in its numerous branches. This is more than ample work for a three years' course, yet if more time is spent in a college, it can only be done by postponing, to an unduly late period, entrance into the practical working world where mere knowledge is not the most

* Professor, College of Engineering, Madras, India.

essential qualification for success. It is extremely desirable that the staff of an engineering college should be composed of men who are not only eminent for their scientific attainments, but distinguished also as experienced and successful practical men. There is a tendency nowadays for Engineering Education to get into the hands of men who have passed all their lives inside the walls of colleges, first as students, then as demonstrators and, finally, as professors. To them, engineering is a science and as such they materially assist in its advancement, but they contribute little or nothing to bringing about a more intimate connection between the colleges and the workshops, which is obviously the direction in which efforts ought to tend. In this respect, America seems to be far in advance of England and, in consequence, American students find it much easier to get started in life, than is the case with English students, who, frequently, after leaving college, have to pay a premium to be articled to an engineer in practice. Mr. Chatterton.

Sir W. H. White.

Sir William H. White, President, Inst. C. E., London, England.—Professor Woodward finished his paper with remarks which embody the most hopeful sign of the times in regard to Engineering Education. He said that when engineers begin to take up the subject of Engineering Education it is likely to be settled. Hitherto professors have been left too much alone; engineers have been too busy with practical work to assist as they ought to have done. Professors have done their best no doubt; but it would be advantageous if no man became a professor of engineering until he had been a practicing engineer, and if no man continued as a professor of engineering for more than a limited time. These may be considered drastic views, but, if adopted, they would remove many existing difficulties. Some professors have remained professors so long that they are out of touch with the practice of engineering. Like everybody else connected with engineering, we, in England, are very much concerned about this matter, and, during the past year (1904), a committee has been formed by the Institution of Civil Engineers, which, it is believed, will do very much toward reaching a solution which will meet present needs. It is really a strong, representative committee of British engineers, in which the Institution of Civil Engineers takes the lead, and to which they have invited engineers from the principal British engineering societies. The committee is as yet only in the earlier stages of the inquiry, but the lines on which that inquiry is proceeding may be briefly indicated. First, a settlement will be attempted as to the preliminary education that ought to be given to those intending to become engineers. Second, an endeavor will be made to indicate the best preparatory training of a practical nature, if any, before students of any branch of engineering enter purely technical institutions.

Sir W. H. White.

In the Institution of Civil Engineers, Civil Engineering is held to embrace all kinds of civil engineers except the military engineers, and it is desired to find some practical course of training which shall be common and useful to those who are hereafter to enter on any branch of Civil Engineering. The committee is, therefore, considering the question: Ought there not to be a common workshop course in mechanical engineering, associated with opportunities of acquiring information in regard to the practical side of electrical engineering?

The proper standard of qualifications for entering purely technical institutions is also to be fixed, and the period of study in technical institutions during which students who are afterwards to specialize can usefully be taught together. At what stage shall they specialize, and to what extent shall specialization go in the institution? Those, briefly, are the lines on which these inquiries are being pursued.

Each of these inquiries represents a great amount of work. There are sub-committees, and these questions are being circulated to obtain opinions from those most competent to judge. It is hoped to obtain from this mass of opinion leading points upon which authorities can agree. In this manner, it is hoped to do something which may be practical and useful.

In England, there is an increasing number of engineering colleges and universities, and, in order to increase their efficiency, we have done our best to ascertain what has been done in the United States, in Germany and in France, in order that some systematic method of Engineering Education may be framed, which can be generally adopted and which will be subject to modification as circumstances change.

One most important fact is commonly overlooked in these discussions. It is this, that we are not dealing with the ideal conditions that should constitute the education of an engineer; we are not providing solely or chiefly for men of genius; the common system must be that which will suit the average man. If the length of human life could be considerably extended, and if all men could be made of one pattern, we might lay down a general practice with regard to the basis of culture.

Culture, the speaker thinks, is a word that is used frequently in a very limited sense. The older forms of culture are still of great value for certain men of certain occupations. There is no justification for wishing to destroy or disturb the older universities, but they require to be supplemented by modern schools. The gymnasium in Germany is still classical, and there is nothing to prevent a man who is trained there from going subsequently to technical schools. Some of the leading engineers in Germany have done this.

But there are men who cannot take a classical course, and yet have the capacity to make splendid engineers, if a suitable scientific and practical training is provided. What has to be done is to provide for them in modern colleges and schools, and not to attempt to force all minds into one mould. Sir W. H. White.

All must recognize that there is a tendency to treat classical education as superior, particularly, because it is the most ancient, but engineers must not fail to face the situation. This is a department of learning belonging to a certain number of men, some of whom may become engineers, but it would be interesting to discover from the gentlemen who do, how much they retain of their classical knowledge, and the extent to which such knowledge has influenced their professional work.

The speaker is heartily in favor of having the education of every engineer embraced in a good thorough general instruction. He does not want to begin to specialize as soon as a boy is out of his cradle, but he is quite convinced that the average boy ought to begin his preparation for engineering not later than seventeen or eighteen years of age, if he is going to do it well. He is also of the opinion that what the Germans do after full inquiry of what was done in England, and what the French do, is the right thing—that a year or two, beginning at the age of seventeen or eighteen, in an actual engineering workshop or the works is an excellent and almost a necessary thing for the average boy. In Germany it is made absolute. Any one who comes out of the preparatory school or gymnasium in that country, with his graduating certificate, has a right to the next thing, the technical, or the classical university. He has it. There is no matriculation beyond. He is qualified at that stage to pass the standard. Then he must go, whether he likes it or not, into the works. He is not admitted to the technical high school until he supplements his graduating certificate with the evidence of his having had the practical training. Then he goes into the technical high school. The speaker believes that is the right course—not for the genius, he is not speaking of the genius.

The average man has to be provided for in schemes of education; the genius is in a class by himself. The late Lord Armstrong was a great engineer. To hydraulic engineering, gun making, explosives and every other branch of engineering that he touched, he added something. Yet he was forty years of age and a practicing solicitor before he began engineering. Armstrongs do not abound; most of us have to get our education and to get it specialized in early life, if we are to do our work in the world.

Engineers must look at this matter in a practical way and for the average youth. There is a tendency to think it is easy to become an engineer. There used to be a saying in England that "the

Sir W. H. White. fool of the family is made a clergyman." Now there is a disposition to make the fool of the family an engineer, letting him go through the university and into the mechanical science course. This is ridiculous. Engineering is not a profession for the incompetent or semi-capable, but for the best men in the world. Comparatively few men are fitted to take leading positions in any profession. With their usual practical good sense, the Germans recognize this fact, and in their technical high schools train men in numbers far exceeding those who can find employment in the higher grades, so as to be able to select a sufficient number of highly competent men who will eventually rise to leading positions. The men who are trained in Germany as engineers, but who never get to the higher positions in the profession, are all the better for that training in whatever walk of life they finally reach, although their ambitions may be disappointed.

As to post-graduate work in England, there is very limited assistance from the Government, and not so much is done as could be wished. Recently it was represented to the Prime Minister that, as elementary education is free in England, while secondary and technical education are aided by the State, that aid might be given also to university education, including the higher engineering colleges.

The Prime Minister replied that he was in entire sympathy with the request, but that no money was available. The matter will have to be dealt with, however, in the public interest.

The most hopeful post-graduate work in England is done in connection with technical colleges. At the University of Manchester, for instance, in the chemical section a few young men who have taken their degrees pursue courses of research bearing on various manufactures. This is probably the way in which this question will be solved. State aid must not be appropriated to special researches, but rather given to institutions, and those in charge must be left free to use the money as may be thought best, taking all the circumstances into account.

Many in England feel that the United States has an enormous advantage in the existence of many generous donors to Engineering Education. In England, in the old days, there were many pious founders who gave money or land to endow education. All over England one finds endowed schools, some of which have been modernized and adapted to present conditions. What is needed in England is not merely extended State aid, but more "pious founders." We have not so many millionaires as the United States can show, but we have many who are in a position to aid the education of engineers, and we trust they will do so in the same generous way as is done here. The matter is most important to the welfare of the British Empire.

Mr. Fletcher.

Robert Fletcher, Assoc. Am. Soc. C. E., Hanover, N. H. (By letter.)—In the opinions and criticisms which the several reviewers have happily expressed, it is more than ever apparent that Engineering Education is a function of so many variables that complete integration is not to be expected. Strong emphasis on some of these variables has been well bestowed: by Professor Van Ornum, on the personal element, in regard to which a complementary view would include the effect of the considerable diversity in the *personnel* of the taught; by Mr. Ray, on the great value of close personal contact between teachers and taught—that good understanding and acquaintance which enable both parties to work more effectively; by others, on the importance of culture and the broad outlook, not only because they enhance the usefulness of its possessor, but are also necessary to counteract the "narrowing" and "falling away" tendency also referred to; and here we might suggest the advantage of having men sufficiently well qualified to teach in more than one department or subject, which would tend to promote the best adjustment and co-operation between the various departments. Educational fads have been rightly deprecated, and such judgment should extend to overdoing along any one line. The value of research work, as a legitimate part of a course of instruction, has again been questioned; and other points, more or less familiar, have been urged and reiterated.

In considering what has been presented, we may reflect that it is easy to set up an impractical ideal; that we must study to reduce the problem to its lowest terms; that our soundest theories and best-devised standards must be modified with the rapid changes in the conditions of working; and that, while Engineering Education must always present various aspects and serve various ends, it may be and should be conducted so as to give thorough and adequate preparation along lines broad enough to produce not only the engineer, but the man of culture and the man of affairs.

Mr. Woodward.

Calvin M. Woodward, Esq., St. Louis, Mo. (By letter.)*—There can be no doubt that Engineering Education is recognized on both sides of the Atlantic as increasingly important to our higher civilization, and as a species of liberal culture of great dignity and worth. It has been the writer's good fortune to be connected with an engineering school for more than thirty years, and he knows what a strenuous life the engineering student leads. He does not mean that it is unlovely or unattractive, or in any way too severe, but it is earnest, serious work. The student atmosphere is bright and happy, yet unlike that in the traditional college, and, on that very account, it is attractive to the great body of American youth.

It was the writer's privilege to point out in his paper a recently added educational feature which adds much and will continue to

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Woodward. add much to the fundamental training of an engineer, namely, manual training. It is not a kind of Engineering Education, it is only a part of the preliminary training in a secondary school, but it is a feature which is valuable to every man, whether he becomes an engineer, a business man, or a lawyer. It is not opposed to the classics, or to any other kind of culture.

Mr. Ray's observation of manual training must have been unfortunate. He must have seen something of manual training falsely so-called. The writer suspects that he is wholly right in his estimate of what he once saw in the New York schools. In one sentence, he gives a picture of a sort of manual training school which the writer should condemn as strongly as he does. He says: "A smart boy * * * may learn more in half an hour by watching a skilful mechanic than a manual training school will teach him in a week." Evidently, Mr. Ray's manual training school had no skilled mechanic, or at least he did not allow the smart boys to watch him. In either case, the fact condemns the school. In a manual training school, properly so-called, every boy, smart or slow, has many half hours every week watching a skilful mechanic, all of whose work is thoughtfully planned to be instructive. Did Mr. Ray ever see a section of boys at a "demonstration" in the tool laboratory of a well-organized manual training high school? Were they not seated in concentric rows about the bench or lathe of the skilled teacher, and intent upon his every word and every act as he went carefully and thoroughly over the what, and the how and the why of the day's lesson? Could a smart boy have a better chance to see and to understand? And when the "demonstration" was over (it lasts from ten to thirty minutes) did Mr. Ray follow the boys to their separate benches or machine tools, and see them put into actual practice the theories and instructions just received?

If he never saw all this, then he never saw a real manual training school. He may have seen little people playing with edge tools, in charge of teachers who did not know how to use them—all trying to make wooden engines or paper boats, and calling it "manual training." If so, the word "fad" was not inappropriate. The writer is reminded of what a skilled carpenter once said to him as he stood watching a division of manual training boys making their first weld in the forging shop.

"You seem much interested in what these boys are trying to do," said the writer.

"Yes, I worked in a blacksmith's shop a year when I was young."

"Then this welding must seem a simple thing to you?"

"Not at all. I never made a weld; I used to be a 'helper,' and 'strike' occasionally, but I was never allowed to try to weld by myself. These boys learn more in a month, working two hours a day, than I learned in a whole year, yet I had to put in ten hours a day."

But when the boy leaves the manual training school he is no engineer. Four years of higher education are still ahead of him. He may get an A. B. or a B. S., with a C. E. or an E. E. in the future, but he will never forget what that skilful mechanic taught him in the manual training school. The number of such boys in the world is increasing rapidly, and out of their abundance the engineering schools are drawing the best material for future engineers. Mr. Woodward.

It is undoubtedly true that the more training and culture a man gets, the more likely he is to lead in the world's work. The writer looks into the coming years, as does Sir William White, with high hopes and great expectations.